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ORDINARY MEETING.

23 February, 1937.

Sir ALEXANDER GIBB, G.B.E., C.B., F.R.S., President, in the Chair.

On the recommendation of the Council, the members present elected by acclamation as an

Honorary Member.

His Royal Highness George, Duke of Kent, K.G., K.T., G.C.M.G., G.C.V.O.

The Council reported that they had recently transferred to the class of

Members.

Hugh Philip Bishop.
William Henry Cadwell.
Charles William Glover.
Henry Alexander McGillycuddy, B.E. (*Royal*).
Edward Mallett, D.Sc. (Eng.) (*Lond.*).
Charles Gordon Sturt, A.F.C., B.Sc. (Eng.) (*Lond.*).

And had admitted as

Students.

Harold Charles Allaway.
James Anderson Morris Baxter, B.Sc. (*St. Andrews*).
Edgar John Brettell, B.A. (*Cantab.*).
Herbert Stanley Brown.
John Marshall Brown.
Edward Noble Ralph Bufill.
Harold Cammack.
Eric Rowland Caslake.
Michael John Paul Casserly, B.Sc., B.E. (*National*).
Richard Neville Coates.
Richard Edwin Collender.
John Douglas Cornwell.
Idris Oke Cottell.
Samuel Geary Cotton, B.A. (*Cantab.*).
Eric Stanley Couchman.
Charles Henry Crombie.
James Cumming, B.Sc. (*Glas.*).
George Eric Ian de Verteuil.
Isaac Theodore Goodfriend.
Reginald Charles William Green.
Henry Halkier.
Arthur Stanley Hall.

John Douglas Hay.
William John Henderson.
Arnold Kalk.
Albert Joseph Keeling.
Sanat Kumar Lahiri.
Owen Douglas Arthur Le Feuvre.
Robert Agnew Liggett, B.Sc. (*Belfast*).
George Cyril Lillicrap, B.Sc. (Eng.) (*Lond.*).
William Henry Maddock.
John Edward Mahoney, B.Sc. (Eng.) (*Lond.*).
Stephen Leonard Mitchell.
Joseph Henry Oakley, B.Sc. (*Birmingham*).
Harold Cyril Page.
Edgar John Parker.
Douglas Cameron Pearson.
William Henry Philpotts.
Geoffrey Whitemore Pickin.
Edgar Snow Potton.
Richard Henry Reiss.
Norman Johnson Reynolds.
Albert James Riley.
Norman Keith Rose.
Robert Stuart Roskams.
Douglas Pickard Scott.
William Richard Scott.
Arthur Sewell.
Malcolm Noel Sharland.
John Charles Lyall Southam.
Alexander Cyril Taylor.
Trevor Jack Thornton.
Frederick William Tindle.
George Barras Treasure.

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

Paper No. 5120.

"West Middlesex Main Drainage."[1]

By David Mowat Watson, B.Sc., M. Inst. C.E.

TABLE OF CONTENTS.

Introduction.

The Author's reason for presenting the following Paper to The Institution is to place on record a brief history of the West Middlesex Main Drainage scheme, and an outline of some phases of the design and construction. "High-pressure" work by such a large staff as was of necessity engaged made it impossible for any one man to be fully conversant with all details, either of design, construction, or methods employed, and the Author has therefore made free use of notes prepared by the leading engineers in a number of departments concerned with the work.

History.

Lying wholly within the valley of the Thames, the County of Middlesex is drained by five main streams (Fig. 1, Plate 1), the Lee, the Brent, the Crane, the Colne, and the Ash, with their tributaries. Separating the Brent watershed from the Lee watershed lies the high ground on which are situated Finchley and Chipping Barnet. From the east of this ridge the natural drainage is into the Lee which joins the Thames below London, whereas the remainder of the county lying to the west drains to the Thames above London. The area known as the West Middlesex Drainage District includes practically the whole of this latter part of the county. It is some 160 square miles in extent and is bounded on the north by a natural watershed ridge, on the west by the river Colne, on the south by the Thames, and on the east by the Finchley and Barnet ridge, and by the London main drainage district.

[1] Correspondence on this Paper can be accepted until the 15th August, 1937.—Sec. Inst. C.E.

After the War West Middlesex began to increase rapidly in industry and population, developing so quickly that the Local Authorities were unable reasonably to keep pace with the provision of public services and in particular of sewerage and sewage disposal.

In June, 1928, Mr. John D. Watson, Past-President Inst. C.E., was invited by the County Council to investigate and report on the whole question. At that time there were twenty-seven sewage-works operated by twenty-two Local Authorities; roughly £250,000 had been spent on them since the War, and a further expenditure of no less than £350,000 was authorized by the Ministry of Health for the same purpose. Mr. Watson's Report was issued in January, 1929, and showed, *inter alia*, that the rivers of the county were becoming overburdened by sewage, and that six of them were discharging their sewage effluent into sources of potable water. The following figures were quoted in the Report and show the growth of the constituent authorities :—

Authority.	Population, 1921.	Population, 1928 (estimated).	Percentage increase, 1921–1928.
Brentford and Chiswick U.D.	57,974	58,300	0·6
Ealing M.B.	89,697	100,000	11
Feltham U.D.	6,326	7,500	18
Hampton U.D.	10,677	13,000	22
Hayes U.D.	5,303	13,500	154
Harrow U.D.	19,468	22,500	16
Hendon U.D.	56,014	89,000	59
Hendon R.D.	17,660	31,500	78
Heston and Isleworth U.D.	46,664	60,000	29
Kingsbury U.D.	2,800	6,000	114
Ruislip Northwood U.D.	9,113	15,000	65
Southall Norwood U.D.	30,261	35,000	16
Staines U.D.	7,329	8,000	9
Staines R.D.	25,063	29,000	16
Sunbury U.D.	6,000	6,500	8
Teddington U.D.	21,216	24,000	13
Twickenham U.D.	34,790	38,000	9
Uxbridge U.D.	12,919	15,000	16
Uxbridge R.D.	10,643	15,000	41
Wealdstone U.D.	13,439	25,000	86
Wembley U.D.	16,191	28,300	75
Yiewsley U.D.	4,845	6,000	24
	504,392	646,100	Average 28

The recommendations expressed were that, since the area under consideration was virtually self-contained, comprising several natural watershed areas, a Drainage Board should be formed and charged with the duty of conveying the sewage from the entire area to the most suitable site available near the Thames (or at most to two sites) and of purifying it to a high standard before discharging it into tidal waters of the Thames below all sources of potable water.

The scheme was well received, and many discussions with Local

Authorities followed. A sewage-works site at Syon Park was favourably recommended in October, 1929, but public opinion was so strong against the utilization of this site that the proposal was abandoned. In January, 1930, the site of the then existing Heston and Isleworth sewage-works at Mogden, although not so suitable in many respects, was chosen as being the next best, in conjunction with a site at Perry Oaks, near Longford, for the disposal of sludge.

The Local Government Act, 1929, gave the County Council itself power to assume the duties of a main-drainage authority instead of forming an *ad hoc* board for the purpose, and in December, 1929, they accepted the task of proceeding with the work. Financial help from the Government in the form of a grant was necessary, however, and the consent of all parties was conditional on such assistance being obtained.

Instructions to prepare Parliamentary plans were issued in July, 1930, and 4½ months later they were deposited with the Bill. About 70 miles of sewers had to be located and surveyed in about 12 weeks; Ordnance maps were quite out of date owing to the rapid development, and extensive surveys had to be made; the book of reference, including over 4,000 names, was prepared; and particulars of the probable points of connexion to the sewers of twenty Local Authorities were investigated.

Twenty Local Authorities were affected by the Bill, and others could have been included had they so wished, but revision of local boundaries later reduced the number of actual authorities constituting the Drainage District, and the authorities concerned to-day are the Boroughs of Brentford and Chiswick, Ealing, Hendon, Heston and Isleworth, and Twickenham, and the Urban Districts of Feltham, Hampton Wick,[1] Harrow, Hayes and Harlington, Ruislip-Northwood, Southall-Norwood, Staines, Sunbury-on-Thames, Teddington, Uxbridge, Wembley, and Yiewsley and West Drayton.

The Government indicated early in 1931 that a grant in aid of the financing of the work would be made on certain terms. The grant was to be based on 75 per cent. of the loan charges on approved expenditure for 15 years, £4,290,000 being the maximum sum ranking for computation of the grant, and the chief conditions attaching to it were that a "substantial start" should be made on the work by 31st October, 1931; that the work should be completed by 1st October, 1935; that 90 per cent. of the labour should be recruited through the local labour exchanges; and that of the 90 per cent., no less than 70 per cent. should be men from the Distressed Areas.

As the preliminary conversations with the Ministry of Labour

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[1] Hampton Wick will participate on the expiry of an existing agreement with Kingston-on-Thames.

had made it clear that speed of construction was one of the Government's chief desiderata, a period of only 5 years for construction, inclusive of design and all other office work, was agreed to with some misgiving. When the time available to complete the work was reduced by the Unemployment Grants Committee to 4 years, the County Council were left without choice.

The Royal Assent to the Bill was given on the 11th June, 1931, and thereafter output from the drawing-office was strained to the utmost. The Parliamentary estimate was £4,525,000 for constructional works, and as there were only 4 years and 3 months to elapse before the grant-earning period closed, the average rate of spending had to be no less than £88,725 per month, a sum sufficiently high to make speed of design and preparation of contract documents a formidable task.

The actual rate of expenditure attained a maximum of £249,000 in 1 month, with an average of £92,400 per month throughout the 4 years.

The following Table gives by inference the pressure on the drawing office and shows the dates of opening of tenders and the amounts earned under the contract.

Contract No.	Date of opening.	Amount: £	Contract No.	Date of opening.	Amount: £
S.1	April, 1931	2,639	S.18	May, 1933	55,406
M.1	July, 1931	28,960	S.19	June, 1933	81,879
S.2	July, 1931	281,234	S.20	July, 1933	110,200*
S.3	July, 1931	299,418	P.1	Sept., 1933	30,494
S.4	Oct., 1931	81,412	M.3	Oct., 1933	715,000*
S.5	Nov., 1931	58,065	S.21	Nov., 1933	129,322
S.6	Nov., 1931	52,844	S.22	Jan., 1934	106,107
S.7	Dec., 1931	237,032	S.23	Jan., 1934	65,025
S.8	Jan., 1932	82,105	S.24	March, 1934	54,384
S.9	Feb., 1932	109,948	S.25	April, 1934	28,672
S.10	March, 1932	115,493	S.26	May, 1934	86,000*
S.11	July, 1932	17,311	M.4	May, 1934	397,000*
S.12	July, 1932	47,524	P.2	June, 1934	147,800*
S.13	Oct., 1932	19,587	M.5	July, 1934	426,000*
S.14	Dec., 1932	81,560	S.27	Nov., 1934	23,969
S.15	Jan., 1933	208,873	M.3b	Dec., 1934	2,076
S.16	March, 1933	368,808	S.29	Dec., 1934	8,192
M.2	March, 1933	191,874	S.28	Jan., 1935	54,200*
S.17	May, 1933	102,994	—	—	—

* Estimated.

The Act made provision, *inter alia*, for :—

(1) The County Council to construct trunk sewers and disposal-works.

(2) Completion to be effected by the 1st October, 1935.
(3) The County Council to make all necessary alterations to local sewerage-systems in conference with the Local Authorities.
(4) The Local Authorities to pay for subsequent local connexions.
(5) Dry-weather flow to be defined as "the rate of 40 gallons per head per 24 hours."
(6) The County Council to be under no obligation to receive sewage-flows greater than a rate of 6 times the dry-weather flow.
(7) Trade-wastes only to be admitted to the system subject to the County Council's conditions.
(8) The County Council to have power to exclude trade-wastes, if injurious.
(9) Effluent from the Council's purification-works to contain not more than 3 parts per 100,000 of suspended matter, and to take up not more than 2 parts per 100,000 of dissolved oxygen, in 5 days at a temperature of 65° F.
(10) The County Council to indemnify Local Authorities against outstanding loan charges on their existing sewage-disposal works, and, if so requested by the Local Authorities, to demolish such works and to clear the sites.
(11) The County Council to be empowered to make agreements with outside Authorities, subject to unanimous approval of constituent Authorities.
(12) Various clauses for the protection of Statutory Companies, etc.
(13) Safeguards against danger to navigation.
(14) Financial clauses, the cost of the work being defrayed by a special rate on the drainage-area.

The constructional requirements involved by the Act may be summarized as follows :—

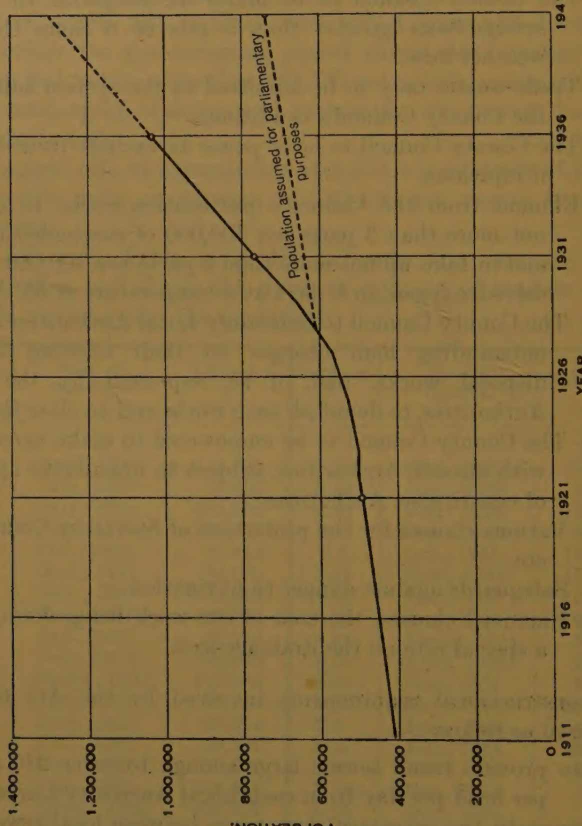
(*a*) to provide trunk sewers large enough to carry 240 gallons per head per day from each Local Authority's area ;
(*b*) to make the necessary connexions between local sewers and trunk sewers ;
(*c*) to make provision for gauging the sewage-flow from each Authority, to ensure the maximum use of the sewers whilst affording fair play to each constituent Authority ;
(*d*) to provide sewage-purification works at Mogden, outfalls to the river Thames at Isleworth Ait, and sludge-disposal works at Perry Oaks.

Main Sewerage.

Population.

Under the provisions of the Act, the onus of allowing sufficient capacity in the sewers rests with the County Council. The aggregate population expected ultimately to reside in the drainage district when fully developed is 2,000,000, and although this figure can safely

Fig. 2.



Population of Drainage District.

be taken for the purpose of designing the purification-works, its distribution over the district is uncertain. For this reason, in the design of individual sewers it was considered essential to allow a margin of safety, which has resulted in an average excess capacity of about 15 per cent. over that required to comply with the bare stipulations of the Act. *Fig. 2* shows the actual population up to

1936, the anticipated increase up to 1941, and the increase anticipated in 1928 for parliamentary purposes.

In the design of trunk sewers, density of population in the gross areas of the districts served is of greater moment than density in more circumscribed zones of building development. Nevertheless, the density of population per acre in various localities of the drainage district is of interest.

	Density of population per acre.		
	Least.	Greatest.	Average.
1921 [Census]	1·4	25	4·8
1931 ,, ...	2·4	25·5	7·5
1936 [estimated]	—	—	10·5
Ultimate densities assumed for design purposes	15	32	23

Water-Supply and Sewage-Flow.

In 1933–34 the average daily consumption of water within the Metropolitan Water Board area was over 40 gallons per head, and the tendency is for this to increase. The maximum rate of flow assumed for purposes of sewer-design as 240 gallons per head per day is not, therefore, excessive. At present only about one-half of the ultimate population exists, so that to make the fullest use of the sewers in the early years a greater discharge than the above allowance is permitted and Local Authorities are allowed temporarily to discharge excess storm-water into the system.

Although neither the sewers nor the purification-works are intended to deal with all the storm-water of the drainage-area, it is interesting to compare the discharging-capacity of the sewers designed on this basis with the rainfall-equivalent. If it is assumed that 6·5 per cent. of the drainage-area is now impervious, it appears that the main sewers are at present able to cope with the run-off to be expected from a rainfall of 0·10 inch per hour falling steadily over the whole drainage-area. Later, as the impervious area increases to, say, 15 per cent. of the whole, this rate of rainfall will be reduced to 0·06 inch per hour. The Ministry of Health formula for the anticipated rainfall indicates a probable maximum of 0·10 inch per hour in this particular case.

Some of the sewers of the Local Authorities are on the combined system and some on the separate system. Of the latter systems it is sometimes advisable to enquire closely; despite claims that it was "strictly separate," one of them was found to discharge during storms 12 times the dry weather flow, another yielded infiltration of ground-water to the equivalent of 22 gallons per head per day,

whilst yet another was suspected of yielding 70 gallons per head per day. Nevertheless, the existing systems are at present not developed sufficiently to make full use of the new trunk sewers.

The discharge of trade-wastes to the sewers was anticipated and encouraged, but the County Council have right under the Act to exclude trade-wastes to be discharged through new or enlarged connexions unless the reasonable conditions of the Council are accepted, and the Local Authority must enter into and enforce an agreement with the manufacturer to that effect.

Geological Information.

Speaking broadly, water-bearing ballast to a depth of 20 feet overlies London blue clay except north of a line drawn east and west through Hillingdon. Tunnelling in London clay is comparatively cheap, easy and fast, so that care was taken to locate the sewers in it so as to avoid unnecessary troubles in ballast. Before embarking on any constructional work, boreholes were sunk at intervals of about ¼ mile along all the proposed sewer-routes, and at a closer spacing where conditions seemed to warrant it. The former spacing usually gave a good general idea of the clay-level, but when the clay cover was not more than, say, 8 feet above the proposed tunnel the minor irregularities of the clay-level demanded a closer spacing of the boreholes during tunnelling. As a result of geological data becoming available subsequent to the preparation of Parliamentary plans, two routes were considerably shortened, because what had been believed to be bad ground was later proved to be quite satisfactory. The savings made by these changes were many times the cost of the borehole contract.

Sewer-Lining.

For the lining of the larger sewers, best engineering brickwork was adopted. The bricks selected were specified to be capable of withstanding 400 tons per square foot, and to absorb not more than 2 per cent. of their own weight after 24 hours' immersion in water. In the case of sewers of diameters less than 3 feet 9 inches the lining usually consisted of concrete pipes, aluminous cement being used in their manufacture to avoid the disintegration to which Portland-cement concrete is liable. The pipes had to withstand unsupported an ultimate axial-line load of 2,000 lbs. per foot length of pipe, applied uniformly along the barrel; they were to absorb not more than 2 per cent. of their dry weight after 24 hours' submersion, and were to show no sign of leakage when subjected to an internal pressure of 6 feet of water for 15 minutes. Such pipes were readily obtainable, and as the cost of a sewer laid with them seldom

exceeded the cost of a sewer laid with ordinary concrete pipes by more than 2½ per cent., their use became economically sound.

Roughness-Coefficients.

Concrete tubes may for a few years have a smoother surface than brick-lined sewers, but over a long period the brick lining will most probably present the better surface, and, at the worst, it is renewable; concrete tubes were therefore considered to have a rougher surface over their average period of life. Flynn's modification of Kutter's formula was used, and the value of n was taken as 0·013 for brick, cast-iron and glazed stoneware, and as 0·015 for concrete tubes.

Shape of Sewer-Sections.

Egg-shaped sewers were considered, but, in general, they were found to be more costly than circular sewers, their extra depth entailed some slight loss of head, and in tunnels driven in poor ground the circular section is better able to resist squeezing. Their advantage in giving a velocity of flow about 10 per cent. higher than the corresponding velocity in a circular sewer at low flows was not deemed to be worth serious consideration, and circular sewers were finally adopted throughout the scheme. Even in the last 5 miles of the Brent Valley sewer, where conditions were exceptional, the circular section proved the most favourable.

Materials and Construction.

The types of sewer employed varied according to the depth and the use of open-trench or tunnel construction. In the case of open trench, apart from a few glazed-stoneware pipes, all sewers up to 3 feet 9 inches in diameter were either cast-iron pipes or concrete tubes surrounded by 6 inches of concrete, according to circumstances. Where the smaller sewers passed through the waterlogged gravel, overlying London clay, cast-iron turned-and-bored pipes were used with success. These gave a dry sewer, were easily laid, required no lead joints, and it was found that the overall cost of a length of sewer of small diameter laid with these pipes was no more than that for cast-iron pipes jointed with lead. For the larger sewers a 4½-inch pressed-brick lining was used. Where the internal diameter exceeded 5 feet 6 inches the concrete surround was suitably reinforced, and for sewers of 9 feet and over in diameter two rings of brickwork were used above the horizontal diameter in order to facilitate the speedy removal of drums without damaging green concrete above. Special grouting bricks were built in at springing-level on some of the larger sewers to ensure a water-tight joint at a point where the brickwork and the concrete of the invert invariably suffer from dirt before the arch can be built.

Sewers in tunnel, if less than 4 feet in diameter, were usually constructed of concrete tubes surrounded by at least 6 inches of concrete. Larger sewers were always lined with brick, whatever other materials of construction were used. Originally the choice available for the construction of larger sewers in tunnel was (i) two or more rings of brickwork, (ii) concrete with brickwork lining, including brick-lined concrete invert and two or more brick rings without concrete in the arch, (iii) cast-iron segments lined with concrete and brickwork. In 1931 there were more than 47 miles of sewer to be designed and constructed in tunnel in little over 4 years, so that there was the danger of asking altogether too much both of the brickyards capable of supplying suitable bricks and of the supply of skilled tunnel-bricklayers, more particularly when the brick-lining of sewers in open trench had to be done concurrently. The schedule of operations was therefore drawn up so that the sewer-bricklaying should be spread over the maximum time.

A factor which materially eased the difficulty of excessive brick-laying came about as the result of the tenders received in July, 1931, for the first two sewer-contracts. In each case the tender of Sir Robert McAlpine & Sons (London), Ltd., was the lowest, and they submitted with each tender a proposal for an alternative tunnel-lining previously patented by them, which consisted of 12-inch rings made up of tongued-and-grooved pre-cast concrete segments, reinforced with a steel hoop in each circumferential joint, the whole being grouted to refusal with cement, and then lined with brick. One of the chief advantages of the method was that the percentage of men from the Distressed Areas employed would be higher, because making segments in a blockyard requires less specialized labour than tunnelling operations. Furthermore, the limited supply of sewer-bricklayers would not be called upon to the same extent. The lowest tenders being accepted, the alternative method of lining became available for adoption, and it was tested by Professor S. M. Dixon, O.B.E., M. Inst. C.E.

A favourable report having been received, construction was started. An elaborately organized blockyard covering about 14 acres was laid out and equipped to supply the necessary concrete segments. Steel moulds were employed, and the concrete hand-tamped and screeded in them. No segment was permitted to leave the block-yard until it was 7 days old, and the concrete was required to withstand 2,500 lbs. per square inch crushing stress at 7 days without sign of fracture. Segments less than 5 inches thick were lightly reinforced for handling. The segments were 12 inches wide, so that driving could be carried out in short lengths, one ring at a time, and at considerable speed. When a length had been excavated, the

steel hoop, consisting of two semicircular bars of from $\frac{5}{8}$ inch to $\frac{7}{8}$ inch in diameter, according to the size of the tunnel, with fish-tailed ends, was inserted in the groove in the leading end of the already erected ring, the segments to form the new ring built up from the invert, and the key block pushed in from the face. Every ring was reeled. The joints were then pointed with cement mortar, and when this had set the lining was grouted to refusal under pressures of from 80 to 100 lbs. per square inch. Special grout-holes ensured that the cement grout found its way not only to the back of the ring but into the joints and around the steel hoops. Good grouting is essential to the success of the system, and therefore various methods were tried, and minor improvements effected as the work progressed. It was found possible to erect the segments with a tolerance of $\frac{1}{2}$ inch in line and level. At places selected at random, where the segments had purposely been broken away, a skin of cement grout, sometimes 2 or 3 inches thick, was found to surround the segments. This must materially increase the strength of the sewer-section, and it accounts for the almost total dryness of the sewers. Inspection proved easy and cheap, and each length was thoroughly tested and made watertight where necessary before bricklayers started the work of lining it. It was possible to carry out the brickwork lining in long lengths, and under these conditions the work is incomparably better than if built in short lengths. After the removal of laggings supporting the soffit of an arch, brickwork tends to settle slightly and to leave a small gap above it; special grouting bricks were therefore built in and cement grout forced in to fill the anticipated void.

Concrete-segment sewers of this form were constructed in sizes from 4 feet up to 10 feet 6 inches in diameter. In two places they were used successfully under compressed air, but in one case the sewer was actually driven in clay, being known to have only a very thin covering of clay between it and the water-logged ballast above. No attempt was made to use segments in ground other than clay, nor was a shield employed with them, though the Author has since inspected short experimental concrete-segment tunnels successfully driven with shields.

Except in the case of very small sewers, cast-iron segments were used under railways, canals, and in special cases where valuable property warranted it.

Shield-Driven Tunnels.

Various sizes of tunnel were driven with shields, and few difficulties were encountered. The largest shield employed was of about 15 feet external diameter for a sewer of 12 feet 9 inches internal diameter,

and the smallest shield was of about 5 feet 5 inches external diameter. The latter shield was too small to afford reasonable working room for miners; furthermore, some trouble was experienced in keeping it on line because only four rams could be accommodated, and because the ratio of length to diameter was too great. In the Colne valley, where the sewer was in very open water-logged ballast, tunnels were required under the Grand Union Canal and the Great Western Railway, both of which are built on embankments. The ballast was too open for chemical consolidation to be effective, and the use of compressed air was impracticable for the same reason, coupled with the fact that there was very small cover available, especially under the canal, where even clay-blanketing over the line of the tunnel could not be carried out without closing the canal. A new type of shield was therefore designed by the Resident Engineer, Mr. H. H. Hunt, Assoc. M. Inst. C.E., in collaboration with Mr. A. R. Kearney, for use in water-logged ballast without compressed air, clay pocketing, or timbering. The front part of this shield is divided by two horizontal tables into three compartments, in which the ballast was caused by fixed sloping shutters to lie at an angle steeper than its natural angle of repose; irregular coning in the compartments therefore caused compression of the ballast. Excavation from the compartments took place only during a "shove," whilst the shield had ram-pressure applied, and the openings through which the ballast was excavated were controlled by hinged doors forming rearward extensions of the tables.[1] The shield was 6 feet 6 inches external diameter, and proved successful, no ground being lost even in loose open ballast, though the "cover" above the shield was in places no more than 7 feet. It was used in three places on this contract and again later on another contract.

Compressed Air.

The highest pressure required at any point on the work was 16 lbs. per square inch, but the more usual pressures were only about half of this. Of the 47 miles of sewers constructed in tunnel, fourteen sections of a total length of 4,450 yards were driven in compressed air.

Sewer-Design.

The onus was placed upon the County Council to accept a maximum volume which depends solely on the population discharging to the trunk sewers at a number of different points. It was thus advisable in certain cases to have some small margin of safety at the head of each branch sewer to allow for the vagaries of future development,

[1] See also pp. 614 and 615, *post.*—Sec. Inst. C.E.

but the time of concentration of storm-water was not taken into account in the design of the sewers, as it was considered possible that, during periods of prolonged rainfall, the full quota of sewage could be received at the works from every district simultaneously.

Fundamentally, the system is designed to run full, with soffits level at all changes in diameter, but, at the junctions of small sewers with large, special consideration was given to avoid the turbulence, splashing and erosion which inflexible adherence to this principle would cause, the branch sewer being lowered until the levels of the dry-weather flows in the branch and in the main sewer corresponded. To effect this compromise the gradient of the branch was increased in the last section, the diameter remaining unaltered. Where the velocity at dry-weather flow on the increased gradient exceeded $6\frac{1}{2}$ feet per second a further section of the branch sewer, to the next manhole, was used to lessen the gradient. Tilting the branch in this way has no effect on its discharge when the system is running full, because at the lower end near the junction it becomes slightly surcharged, and at other times there is a tendency to increase the discharge. If the flow in the main sewer rises more rapidly than in the branch, the latter is temporarily backed-up, but usually not farther than the first manhole, and this length is scoured out when the flow in the main sewer returns to normal.

Any diminution in the velocity of the sewage in an outfall may cause silting, and nuisance. Normally, therefore, the dry-weather-flow level in an outfall-sewer should be such that the sewage is discharged freely into the purification-plant without any backing-up of the sewer. If this condition is fulfilled in the case of a large sewer there must be considerable loss of head in times of storm at the end of the sewer, equivalent to roughly three-quarters of the diameter of the sewer. Only a part of this loss is normally required at the works for such purposes as storm-water separation, and the remainder occurs in the sewer itself, being visible as "draw-down." As the last length of the sewer cannot run full it is possible, subject to limitations, either :—

(*a*) to lower the soffit of the sewer to make it correspond with the water-level at maximum flow ; or

(*b*) to reduce progressively the size of the sewer. In this case the invert-gradient may be varied considerably—it can even rise, within the limits of the dry-weather-flow conditions, towards the outfall without affecting the velocity.

The one theoretical limitation to these reductions in size is the approach to the conditions of critical depth at the outlet. For any given rectangular channel the critical depth can be determined with

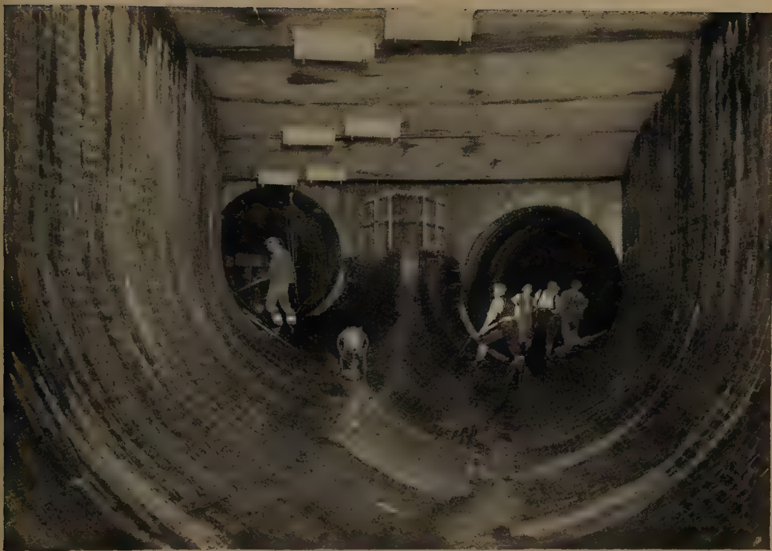
reasonable accuracy, but the critical depth in circular sewers is less certain. The practical effect was to reduce the diameter of the Brent Valley main sewer by 3 inches, at a point 2 miles from the outlet, and greatly to reduce the size of the outfall near the purification-works, alternative (*b*) being used with a constant invert-gradient. The maximum discharge of the high-level sewer was 826 cusecs and the minimum 48 cusecs, the corresponding velocities being 8·1 and 2·1 feet per second respectively. The last 250 feet of the sewer has a circular invert 12 feet 9 inches in diameter, vertical sides above the springing, and a flat roof. At the upstream end of this section there is a venturi-section, the roof tapering towards the invert to enable the sewer to pass under the Duke of Northumberland's river, the bed of which forms the roof of the sewer. Above this point the section becomes a 12-foot 9-inch diameter circle for about 2,000 feet, as far as the junction of the Brent Valley and Bath Road main sewers. In this section two 12-inch diameter sludge-mains are slung in the soffit just clear of the sewage-level under conditions of maximum discharge. The invert-gradient from the junction-manhole to the outlet is 1 in 6,410. The maximum depth of sewage at the outlet is 9·7 feet, increasing to 11 feet at the junction-manhole, about 10 inches below the theoretical maximum discharge-level of the sewer at that point.

Manholes.

A rough rule adopted for the spacing of manholes was 50 yards per foot of diameter up to 6 feet, with a minimum of 80 yards. In the case of larger sewers, the manhole spacing was normally about 400 yards with a maximum of 450 yards. Manholes were built at each junction, and, in the case of small sewers, on each bend. The first few feet of brickwork sewers above and below manholes on tunnelled sewers were strengthened by an additional ring of bricks. Where manholes were constructed with brick inverts two rings of brickwork in the invert and benchings were provided. A concrete foundation of a manhole used for months as a working shaft has usually suffered to such an extent that the most economical way of completing it is not to "doctor" it, but merely to clean it and leave the bricklayer to finish up. This he can do if allowed a backing ring in addition to a lining ring. Reinforced-concrete manholes proved cheaper than brick, and on all later contracts their roofs were reinforced only with rolled-steel joists for simplicity.

Sewer junctions were formed by bell-mouthed chambers giving an easy intersection with the main sewer; a typical junction-manhole is shown in Figs. 3, Plate 1 and *Fig. 4*. The complicated brickwork inverts of junction-manholes invariably cause slow, difficult and

Fig. 4.



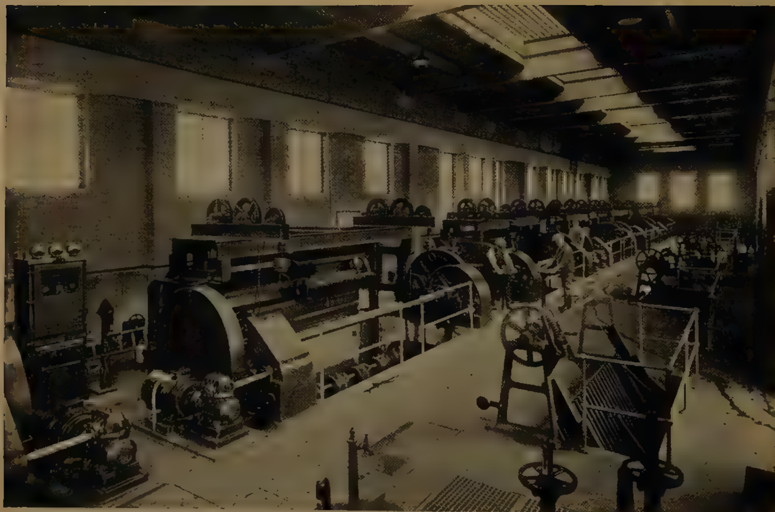
CONSTRUCTION OF JUNCTION-MANHOLE.

Fig. 17.

AERIAL VIEW OF MOGDEN PURIFICATION-WORKS.

5120.

Fig. 19.



SCREEN-HOUSE.

Fig. 20.

CAST-IRON STANDING-WAVE FLUME.

expensive work. To simplify this work the following method was adopted to eliminate the cutting of bricks at junction-manholes. The springing-levels of the three sewers were made, if possible, to differ by a multiple of 3 inches, and the springing-courses of the largest sewer were continued along the benchings horizontally on each of the outer walls of the manhole until they met the eyes of the other sewers. These courses formed a separate datum on each wall, and course after course was marked out below them along the circumferences of the bottom quadrants of the two sewers concerned. The effect of this was to cause each course to follow a continuous spiral, but without break of line or bond, until eventually these spirals intersected on the nose of the cut-water with the corresponding courses that had been similarly marked off from the datum in the opposite wall of the manhole. Each course stopped one brick-length short of the nose (*Fig. 4*), and the actual cut-water was formed of aluminous cement concrete. Thus there was only normal cropping of the bricks on the inside of the curves, and there was no intersection-work.

Where the sewers were built of concrete pipes the invert and benchings of the manhole were provided with an aluminous-cement granolithic lining 3 inches thick, keyed into the Portland-cement concrete underneath; this was found to be more satisfactory than an aluminous-cement rendering, and actually cheaper.

Deep sewers and large sewers of circular section, in which the depth of flow is seldom less than 2 feet, must be entered with considerable caution; platforms to accommodate at least two men and their tools are therefore provided above the highest sewage-level.

Shafts for shallow manholes were built of brick and were 2 feet 7½ inches square. When a shaft was over 20 feet deep it was found economic to construct it of cast-iron flanged pipes 2 feet 9 inches in diameter, platforms being formed where required to divide the climb into stages not exceeding 30 feet, by inserting 4-foot diameter pipes 6 feet long set eccentrically to the shaft, over which a hinged grating was placed. Shafts only 2 feet 6 inches in diameter are uncomfortably small, but 2-foot 9-inch shafts have proved satisfactory.

In certain built-up areas where it was desirable to reduce constructional noises to a minimum, the number of working shafts was halved, and after tunnelling was completed side-entrance manholes were built (*Fig. 5*, p. 478), the shafts being bored—a comparatively noiseless operation. Cast-iron tubbing was used successfully in some instances for sinking manhole-shafts.

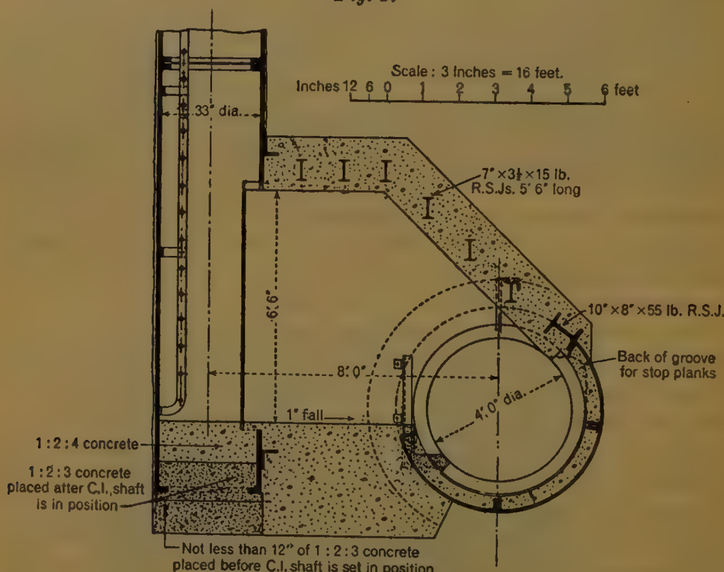
Sewer-Ventilation.

Proper ventilation can be maintained naturally both by convection-currents and by the breathing caused by fluctuating sewage-levels;

the more outlets there are, the better is the ventilation and the less the volume of potentially objectionable air discharged at each outlet. Natural ventilation caused by open manhole-covers and 12-inch air-inlet shafts spaced about 240 feet apart has therefore been relied on. Wherever possible manhole-covers and air-inlets were placed in the carriageway so as to take advantage of the diffusion provided by passing traffic and to minimize the chance of pedestrians standing over them.

The air-inlets to sewers constructed in tunnel were bored and lined with 12-inch cast-iron pipes. Most of them were sunk

Fig. 5.



BORED SIDE-ENTRANCE MANHOLE.

vertically, but many were put down at an angle, in one case as much as 15 degrees out of the vertical. A tolerance of 3 inches in the direction of the sewer and 8 inches in the direction of the slant (or across the sewer) proved quite sufficient for holes 50 feet deep. In open-trench work a flexible joint was provided at the base of the inlet to permit movement during settlement of the back-fill, lead joints proving satisfactory.

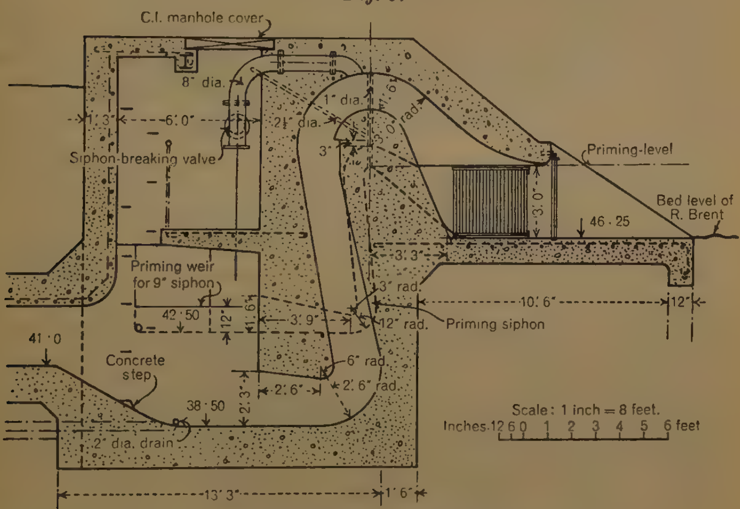
Siphon Spillways on Sewers.

The main outfall, or high-level sewer, passes under the Duke of Northumberland's river at the sewage-purification works at Mogden;

the general ground-level of a great part of the works may be taken as 21·0 O.D., and the water-level of the Duke's river, which passes through the middle of the site on an embankment, is about 28·5 O.D., so that if the river overflowed its banks it could flood a large part of the site. As an emergency relief a small siphon spillway was built, discharging into the high-level sewer where it crosses under the river. The siphon is capable of discharging 80 cusecs and its cost was £50, which compares favourably with that of a penstock of the size required, and it has the advantage of coming into operation automatically when the water-level in the river reaches 32·0 O.D.

Another siphon spillway to come into operation in cases of

Fig. 6.



River Brent Flood-Relief Siphon.

emergency is located in front of the grit-chambers, and will act only if the level in the sewer rises too high owing to the presence of river flood-water or if a grit-chamber inlet-penstock fails to open. Under this condition the excess will be discharged direct to the storm-water tanks.

At two other points on the sewers siphon spillways were constructed to discharge flood-waters to the sewers on those occasions when they are capable of affording relief. One (*Fig. 6*) is situated on the river Brent near Ruislip Road, Greenford, and another on the Wealdstone brook in the old Wembley Exhibition grounds, the former with a discharging capacity of 250 cusecs, and the latter 70 cusecs. Economically it would be wrong to have flood-water

lying in a valley while a large sewer below it was not running full, and until the ultimate development of the county is realized the spare capacity in the sewers must be very considerable. Flood-water will normally reach a given point down the valley more rapidly through the sewers than it will by way of the river, so it may be expected that the incidence of the peak flow in each due to a storm will differ, the sewer flow falling away from its maximum while the river is rising. Apart from the relief for flood-waters afforded by the siphons, it has to be remembered that before the sewers came into use all the sewage and storm-water they now carry were being discharged by the stream, so that the full capacity of the sewer is in itself an addition to the discharging-capacity of the stream.

Inverted Siphons.

In several places in the Brent valley, inverted siphons were required where the sewers crossed the river Brent or the Grand Union canal. The particulars of the principal siphons are as follows :—

Location.	Length: feet.	Maximum capacity: cusecs.	Sections provided.	Dry-weather-flow velocity: feet per second.
Brent Valley main sewer .	130	323	two 26-inch. ,, 33-inch. ,, 6 feet by 3 feet.	4
South Ealing branch sewer .	1,150	32	one 16-inch. ,, 27-inch.	2·9
Hanwell . . branch sewer .	850	11	one 12-inch. ,, 20-inch.	1·8

In every case the desiderata were as follows :—

(*a*) There should be more than one pipe.

(*b*) The upward slope from the lowest point of the siphon to the outlet should be comparatively flat.

(*c*) The pipes should come into operation one at a time, so that before the second pipe came into action the velocity in the first should be such as to keep it clean. Similarly, when the third pipe came in, the velocities in the first two should be sufficient to flush them.

(*d*) It should be possible for every pipe to be shut off as required, so that by closing all pipes except one a high velocity could be produced in any particular pipe to scour it.

(e) Scour-valves should be placed at the lowest points of the pipes so that they could be emptied one into another by means of a pump, and obstructions forced out under pressure.

(f) The dry-weather-flow velocity should, if possible, be not less than 2 feet per second.

If the daily dry-weather-flow velocity in a single-pipe siphon is 2 feet per second, the velocity about midday will rise to, say, $3\frac{1}{2}$ feet per second, while during the night it will fall to about 1 foot per second. In times of storm, if the siphon is to discharge six times the dry-weather flow the velocity will rise to about 12 feet per second. If x denotes the friction-loss at the rate of dry-weather flow, the loss at six times the dry-weather flow will be $36x$. If, however, two pipes are provided, one of twice the area of the other, the loss at six times the dry-weather flow will be reduced from $36x$ to less than $4x$. Moreover, not only is the second pipe a safety measure in case the first should get blocked, but with two pipes the dry-weather-flow velocity in the smaller one is considerably increased and the likelihood of it becoming choked with silt is thus reduced.

When more than one pipe was used to comply with condition (c), the entry to each of the other pipes was set at an appropriate level, so that each pipe carries its full quota, and is being flushed, before another comes into action. The outlets were also set at different levels so that there is no tendency for solids to be carried back into the pipes that are not working. In the case of the Brent siphon the dry-weather flow estimated at the time of the design was little more than one-third of the ultimate, so that special care was needed to prevent silt collecting in it. Two extra pipes were therefore used, four cast-iron pipes carrying the first three times the dry-weather flow and the two culverts taking the remainder. An overflow to the river is provided merely as an emergency measure, but it is unlikely that it will ever come into operation.

In two places where siphons passed under the Grand Union canal, a cast-iron tunnel was necessary. Although space was cramped, the two pipes were laid in the tunnel on concrete chairs.

Working in Tunnel or Open Trench.

As the scheme progressed contractors showed more and more favour to tunnelling as opposed to open-trench work, wherever possible, because it eliminated heavy road-charges, and the cost and delay of diverting water, gas, and other mains; moreover, less annoyance was caused to the travelling public and to road-frontagers. As far as possible contractors were given the option, when tendering,

of using whichever method suited them, but it appeared that even in open country, provided the heading was in clay or other good material, most contractors preferred to tunnel where the cover was as little as 12 to 15 feet on account of numerous service mains.

In some areas open-trench work was out of the question. For instance, along Brentford High Street for a length of several miles there was great difficulty in finding sites for working shafts and even for 12-inch air-inlets, and side-entrance manholes with long passages had to be designed so that the shafts could be located in side streets (Figs. 7, Plate 1).

Steel Sheet-Piling.

Owing to the water-bearing gravel encountered practically throughout the area, and to the proximity of numerous services in roadways, steel sheet-piling was required at most shafts, and in many lengths of open-trench excavation. It had often to be left in to prevent collapse or damage to service mains and structures. Contractors invariably entered a price equivalent to the value of new piles in the schedules for piling left in, and were naturally loath to draw any piling. Subsequently, it was laid down in the specification that only £6 per ton (a second-hand price) would be paid for it, and this stipulation appeared to work satisfactorily and fairly.

Infiltration.

As a result partly of the careful choice of materials and partly of the care taken with workmanship, infiltration was reduced to a minimum. Typical results were :—

(*a*) 8,950 yards of 6-foot 6-inch, 6-foot 9-inch, and 7-foot sewer (concrete segments and bricks), constructed mostly in clay, yielded 1,320 gallons per day (0·92 gallon per minute).

(*b*) 1,800 yards of 6-foot sewer, constructed in water-logged ballast, with cast-iron segments and under compressed air, yielded 60 gallons per day (0·04 gallon per minute).

(*c*) 3,100 yards of 18-inch and 2-foot 6-inch sewer (concrete pipes), constructed in average ground, yielded 285 gallons per day (0·2 gallon per minute).

In two cases where a shallow sewer had been constructed in tunnel beneath an old ash-tip, the sulphur-laden water from the ashes attacked the joints (2 : 1 cement-mortar) in the outer rings, and finally leaked into the sewer, forming stalagmitic growths on the

invert. The attack in each case was quite local, and occurred before the sewers had been brought into use.

Connexions to Trunk Sewers.

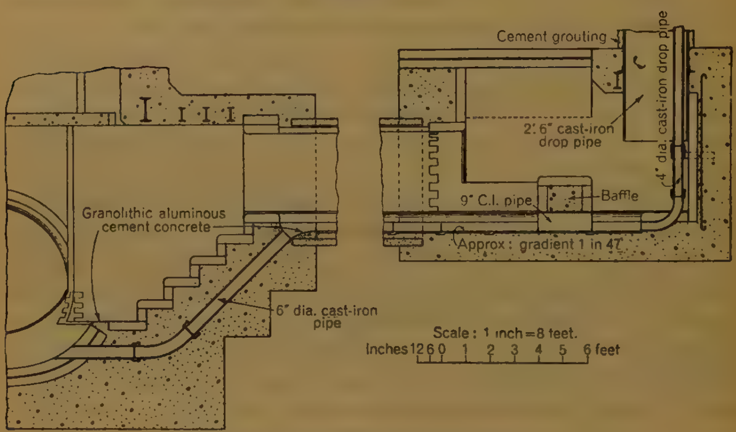
When the scheme was originally contemplated, its fundamental idea was the linking up of local sewerage-schemes by trunk sewers laid to the focal points—the then-existing sewage-works—of the sewerage-systems of the Local Authorities. As the scheme materialized, however, it became obvious that from a broader point of view, considerable general advantage could be obtained by a rather broader interpretation of the basic requirements. Connexions between Local Authorities' sewers and the County Council's trunk sewers were finally agreed on at all rational points of outfall in each district. In practice this resulted in a total of about 130 connexions, the sizes ranging from 6 inches to 6 feet in diameter, and the flows intercepted ranging from less than 1 cusec to over 200 cusecs. The basis adopted represents a compromise between, on the one hand, the acceptance of sewage at only the sites of the Local Authorities' sewage-works, and, on the other hand, the provision of an unlimited number of connexions throughout the new system. Additional connexions will not in the future be allowed, save in most exceptional circumstances, because it is of the utmost importance that the County Council should safeguard its ability to administer the system fairly as between sixteen contributing authorities on the basis of the volume quota laid down under the Act.

Since long lengths of the trunk sewers were constructed in tunnel down to 60 feet below ground, an important problem involved was the design of backdrops. The destructive effect of flows at backdrops was assessed by means of an empirical formula, and was assumed to vary as Qh^2, where Q denotes the maximum flow in cusecs, and h the difference in feet between water-surfaces in the two sewers when both are full. Where $Qh^2 = 0$ to 100, a simple glazed-stoneware pipe backdrop was used; where $Qh^2 = 100$ to 1,500, a simple cast-iron pipe drop was used with the bottom bend concreted in; where $Qh^2 = 1{,}500$ to 10,000, a cast-iron bend with a granite recess at the base or a renewable cast-iron bend was used; and where Qh^2 exceeded 10,000 or where Q exceeded 20, a special cascade or other device was used. The aim was to destroy unwanted energy with the minimum breaking-up of sewage-solids. The two objects are really incompatible, but easy stream-lines for normal-flow conditions, combined with the maximum reduction in velocity with big flows, were aimed at, and wearing surfaces were either made so hard as to be virtually indestructible or were designed to be easily renewable. On large

cascades a by-pass was provided to carry normal flows, the cascade only operating under storm conditions.

Many designs and methods of construction were tried (*Figs. 8, 9, 10, 11*, and *12*). With few exceptions, drop-pipes were of cast iron, and where 90-degree bends were used at the base they were of class "D." Radial junctions were used instead of bends in several cases, one leg of the junction forming a water-cushion. Aberdeen granite facings were also used at the base of many drop-pipes. Cascade-drops were

Fig. 8.



BACKDROP—BY-PASS CASCADE.

always brick-faced, but the construction varied, the following types being all employed :—

(*a*) Rectangular reinforced-concrete section for open-trench construction.

(*b*) Concrete-segment tunnel on ramp (1 in 2½), brick-lined with stone-slab baffles in invert (*Fig. 10*, p. 486).

(*c*) Spiral cascade, brick-lined concrete walls; reinforced-concrete steps, brick-faced (*Fig. 11*, p. 487).

(*d*) Straight cascade, cast-iron steps (*Fig. 12*, p. 487).

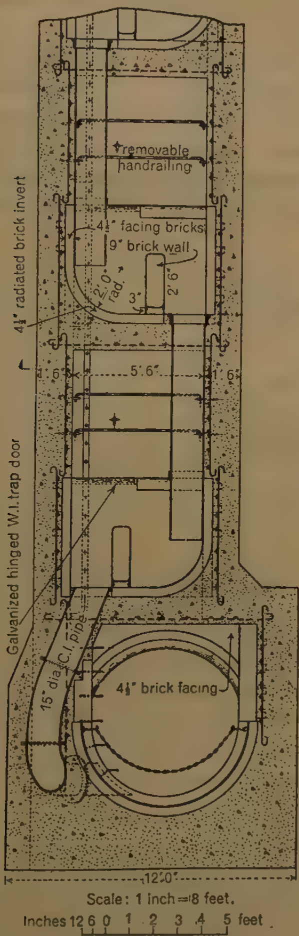
A composite connexion embodying drop, by-pass, and cascade is shown in Figs. 13, Plate 1.

Gauging.

The necessity for gauging the discharge of sewage and storm-water intercepted by trunk sewers arises from the condition laid down by the Act that the County must accept into its sewers a flow equivalent to a maximum of 240 gallons per day per head of contributing

population. The trunk sewers are, as previously stated, designed on the basis of ultimate population, but even so it would be possible for constituent districts to exceed their quota at the expense of others using the same sewer. For administrative purposes it is therefore

Fig. 9.



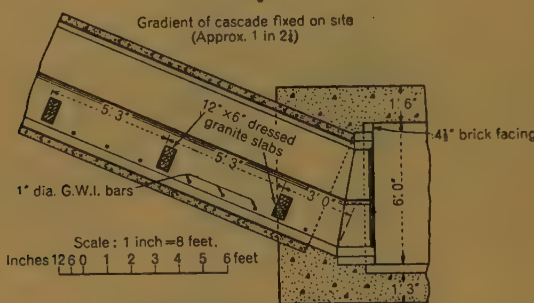
Backdrop—Pipe Cascade.

necessary to obtain records to show what total discharges and maximum rates of discharge the intercepting-sewers are being called upon to carry. Although not essential, it is highly desirable on a system of sewers of this character and magnitude that there should be available as much information as possible correlating data of local

storms and flows in the sewer, thus providing means for research investigation. To make the system as complete as possible, main gauges were installed on trunk sewers at key points, such as boundaries of constituent districts; smaller gauges on important connexions; subsidiary gauges on minor connexions, where a gauge may be required on occasions; and depth-of-flow recorders on the trunk sewers.

There are eleven main gauges of the standing-wave-flume type, with continuous recorders housed in cast-iron boxes above ground. The twenty-two main-branch gauges are similar to the main gauges, but generally of smaller capacities, and the nineteen subsidiary-branch gauges are standard-pattern flumes built into the chambers. Portable interchangeable instruments can be housed in the chambers

Fig. 10.



BACKDROP—RAMP CASCADE.

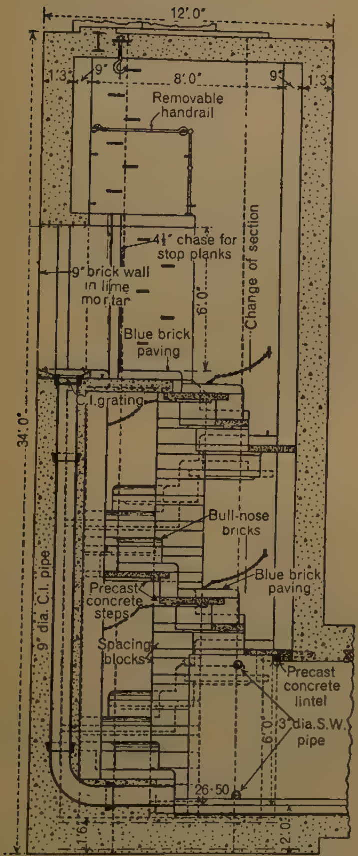
below ground, and are intended only for periodic routine gaugings of the flow in smaller branches. Monthly records of the depth of flow in the main sewers are provided by thirty depth-of-flow recorders operated from float-wells recessed off the inverts of manholes.

Standing-wave flumes were selected in preference to other means of gauging for the following reasons:—

(*a*) Weirs cause loss of head and deposition of solids in the stilling bay.
(*b*) Venturi-meters are unsuitable on gravity sewers.
(*c*) Standing-wave flumes can operate with a loss of head of little more than one-seventh of the upstream depth.
(*d*) Retardation of flow on the upstream side of the flume is practically eliminated.
(*e*) The standing-wave type needs only one measurement, the upstream depth, and the formula is well established.

Rounded-invert flumes were developed to improve further the velocity-conditions at low flows, and were tested by means of models

Fig. 11.

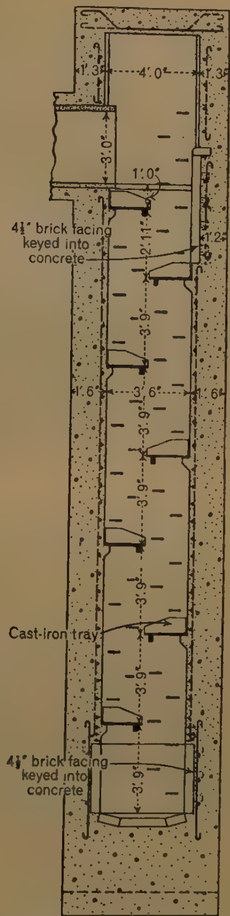


Scale: 1 inch = 8 feet.

Feet 0 1 2 3 4 5 10 feet

BACKDROP—SPIRAL CASCADE.

Fig. 12.



Scale: 1 inch = 8 feet.

Inches 12 6 0 1 2 3 4 5 feet.

BACKDROP—STEP CASCADE.

by the makers, and at the City and Guilds College under Professor S. M. Dixon, and it was found that the following accuracies could be relied upon:—

Full flow to ½ flow	within	±2	per cent. of the correct flow.
½ „ „ ⅕th flow	„	±3	
⅕th „ „ $\frac{1}{10}$th „	„	±5	
$\frac{1}{10}$th „ „ $\frac{1}{30}$th „	„	±6	

Not only was this accuracy considered adequate, but the loss of accuracy on the lower flows was less serious than the reverse would

have been because it is the large flows, approaching permissible limits of discharge to the County's sewers, which are of greatest importance.

The throats of all flumes are of stainless steel, but the approach and outlet "cones" vary according to size. On sewers of over 4 feet in diameter, the "cones" are brick-faced, while the standard pattern, passing only 10 cusecs maximum, was built in special iron castings backed with concrete, and all others were of concrete faced with aluminous-cement granolithic concrete. Approach-channels of U section were designed with lengths of from six to ten times the throat-width. The channels of the larger flumes were formed in concrete, plain or reinforced, inside an enlarged circular tunnel section. Special cast-iron U-section approach-channels were found to be cheaper than concrete for the small flumes.

Float-wells are connected to the approach-channels by slots 2 inches wide, and are either large enough for a man to enter and clean out, or are provided with a scour-valve so that they can be flushed out.

Control of Flow from Constituent Districts.

It was estimated that local sewerage-systems, with few exceptions, could not discharge the maximum flow allowed by the Act, and that the capacity available in the County's sewers in most cases is probably about ten times the present dry-weather flow. There is, therefore, no immediate need for positive control of flow, because by watching the records of the gauges ample warning will be given of any need for control, and the County Council can then instal control penstocks. As, however, the flows from the constituent districts are not controlled, storm-water overflows are installed as precautions near the head of several of the branch interceptors. The principle generally adopted was to use an orifice on the sewer, with the weir set some feet above the soffit of the sewer. In this way the flow passed down the sewer can be calculated fairly closely, because variation of head over the weir is only a small proportion of the head on the orifice. In two instances, where the required orifice was smaller than desirable for normal conditions, a larger orifice was installed and an automatic flap fitted to close and reduce the orifice only when the flow exceeds six times the dry-weather flow. Up to that rate of discharge a clear run through is provided without restriction.

Ground-Water Intakes.

In the Colne valley, where the present development is small compared with the ultimate development, certain sewers are laid to

gradients such that the velocity of dry-weather flow in early years will be less than $1\frac{3}{4}$ foot per second. Provision was therefore made for the controlled admission of ground-water, "Halls-gate" penstocks operated by floats being installed so that the flow of ground-water is cut off when the level rises above half-depth in the main sewer.

Rates of Construction in Tunnel.

Figures of average rates of progress must necessarily be approached with caution, because they are influenced by many factors, and are probably never strictly comparable. Those given below represent rates of sewer-construction in tunnel after the contractor had overcome initial difficulties and was making steady progress.

Diameter.		Progress in 24 hours.
10 feet 6 inches	Concrete-segment sewer, complete except for brick lining	8 feet.
9 feet	,, ,, ,, ,,	12 feet.
6 feet 6 inches	,, ,, ,, ,,	12 feet.
5 feet	,, ,, ,, ,,	12 feet.
10 feet 6 inches	Brick sewer complete	7 feet 6 inches.
6 feet	,, ,, ,, ...	9 feet.
4 feet 6 inches	,, ,, ,, ...	7 feet 6 inches.
4 feet	,, ,, ,, ...	9 feet.
3 feet 9 inches	Concrete-pipe sewer (mining and timbering only)	10 feet.

The following rates refer to cast-iron segment tunnel, exclusive of concrete or brick lining, the diameter given referring to the internal flange diameter of the segment and not to the finished sewer.

Diameter.		Progress in 24 hours.
12 feet	Cast-iron-segment tunnel (shield-driven) .	15 feet.
6 feet	,, ,, ,, in compressed air with clay pocketing .	10 feet.
6 feet	,, ,, ,, without clay pocketing . .	18 feet 4 inches.
5 feet 8 inches	,, segments and Hunt-Kearney shield in free air	10 feet.

Sewer-Costs.

The cost per linear yard of sewer constructed in open trench depends as a rule principally on the cost of excavation, which varies too greatly to be conveniently tabulated. Approximate averaged

costs per linear yard of the sewer-sections, exclusive of excavation are, however, given below :—

Diameter.		£	s.	d.
2 feet	Concrete pipe with concrete surround	2	3	0
2 feet	Cast-iron spigot-and-socket pipe without concrete surround	2	10	0
2 feet 6 inches	Concrete pipe with concrete surround	3	3	0
3 feet	" " " "	4	0	0
3 feet	Cast-iron spigot-and-socket pipe without concrete surround	4	15	0
3 feet	Cast-iron spigot-and-socket pipe with special beaded joint with lugs, without concrete surround	6	0	0
3 feet 6 inches	Concrete pipe with concrete surround	5	0	0
4 feet	Brick and concrete sewer	6	10	0
4 feet	Cast-iron spigot-and-socket pipe without concrete surround	7	15	0
4 feet	Cast-iron pipe with special beaded joint and lugs, without concrete surround	8	5	0
4 feet 6 inches	Brick and concrete sewer	7	0	0
6 feet 6 inches	Brick and reinforced-concrete sewer	15	0	0
7 feet	" " " "	15	10	0
10 feet	" " " "	23	0	0
10 feet 6 inches	" " " "	24	0	0
12 feet 9 inches	" " " "	32	10	0

The following costs per linear yard of sewers constructed in tunnel include excavation, and, in the case of the concrete-pipe sewers include timber left in the heading, and the concrete back-filling to that heading :—

Diameter.		£	s.	d.
1 foot	Concrete-pipe tunnel	4	16	0
1 foot 9 inches	" " "	6	6	0
2 feet 6 inches	" " "	7	5	0
2 feet 9 inches	" " "	8	2	0
3 feet	" " "	8	0	0
3 feet 9 inches	" " "	11	0	0
4 feet	Two-ring brick tunnel	12	0	0
4 feet 6 inches	" " "	13	15	0
5 feet	" " "	15	0	0
5 feet 6 inches	Three-ring brick tunnel	22	0	0
6 feet	" " "	23	0	0
6 feet 6 inches	" " "	24	0	0
7 feet	" " "	25	0	0
7 feet 9 inches	" " "	27	10	0
9 feet	" " "	34	10	0
10 feet 6 inches	Four-ring brick tunnel	43	0	0
4 feet	Cast-iron-segment tunnel [1] lined with brick and concrete	17	15	0

[1] The diameters of the cast-iron-segment tunnels given here refer to the internal diameter of the finished sewer.

Diameter.		£	s.	d.
4 feet 6 inches	Cast-iron-segment tunnel[1] lined with brick and concrete	19	15	0
4 feet 9 inches	,, ,, ,, ,,	20	15	0
6 feet	,, ,, ,, ,,	28	0	0
6 feet 6 inches	,, ,, ,, ,,	33	10	0
7 feet	,, ,, ,, ,,	41	0	0
10 feet	,, ,, ,, ,,	49	0	0
10 feet 3 inches	,, ,, ,, ,,	50	0	0
11 feet	,, ,, ,, ,,	55	5	0
11 feet	,, ,, ,, ,, (shield-driven)	51	15	0

Prices of finished sewers of concrete-segment construction may be taken as approximating to prices of brick sewers of the same internal diameters. The costs of the completed sewer given above, in almost every case, represent an average of the three lowest tenders for the sewer concerned, and do not include manholes, engineering, legal charges, or inspection. In each case due allowance has been made for the incidence of overheads so as to make the figures comparable, as far as possible.

Prices on the earlier contracts were considerably higher than those on the later ones, this being particularly noticeable as regards the cast-iron-segment sewers. Working in compressed air usually added about £10 per linear yard to the cost of tunnelling, to which must be added a lump sum as the cost of the compressed-air plant. The use of a shield increases the cost of driving in the case of comparatively small sewers, and tends to decrease it in large ones, the purchase-price of a shield being considered separately.

The ratio of the cost of manholes to the total cost of the sewer itself depends greatly on the depth to the invert and on the diameter of the sewer. The following Table gives some typical examples :—

Diameter of sewer.	Depth to invert: feet.	Cost of manholes on a given length of sewer compared with total cost: per cent.
9 feet	45	9
7 feet	33	6
4 feet 9 inches	50	16
4 feet 6 inches	25	10
3 feet	53	18
3 feet	14	8
2 feet	12	16
1 foot	13	17

[1] The diameters of the cast-iron-segment tunnels given here refer to the internal diameter of the finished sewer.

These prices are exclusive of sheet-piling left in. The cost of bored air-inlets 1 foot in diameter may be taken as 32*s.* 6*d.* per linear foot, complete.

Costs of Gauging Installations.

The circumstances varied so much at the points of gauging and connexion that it is difficult to give any clear conception of cost without setting out details of each case, but the following very general figures may be of interest :—

The average cost of sixteen main gauging stations on the trunk sewers, extra over normal construction and exclusive of instruments, was £290 each. Instruments cost about £80 each.

The average cost of thirty-eight main-branch connexions, including gauging apparatus but excluding instruments, was £475 each, and the instruments cost £75 each.

The average cost of one hundred and seven subsidiary branch gauging stations where no instrument was installed was £216 each.

The average cost of thirty-three level-recording stations on main sewers was £16 each, exclusive of instruments, which cost £52 each.

Effluent-Conduits and Outfalls.

Isleworth Ait is situated in the river Thames about ¾ mile from the nearest point of the Mogden works. It is separated from the Middlesex bank of the river by a channel which is nearly dry at low tide, but is about 50 yards wide at high tide. On the eastern side of it is the main channel, which is now about 80 yards across to the Surrey bank, having been widened by about 12 yards during the construction of the outfalls. The Ait itself is about 600 yards long with an average width of about 80 yards, and its eastern bank was apparently being eroded at the rate of 1 foot per annum. Vegetation ran wild on the island, and spring tides frequently submerged it. The County Council acquired the island, and with it the Heston and Isleworth sewage-works outfall situated at the southern end.

Richmond Corporation and the Port of London Authority caused protective clauses to be inserted in the Act, and plans were subject to their approval. The Port Authority was empowered, at any time in the future, to require the removal of any works, should it decide to widen or deepen the navigable channel of the river or the backwaters of Isleworth Ait. The main tunnels were therefore kept sufficiently low for the crown to remain well covered if the backwater-channel were ever dredged to navigable depth ; further, each of the twelve outlets is provided with two top sections, each 1 foot 6 inches deep, which can, if necessary, be removed later. All steel sheet-

piling was cut off below the lowest removable section. The Richmond Corporation required that the highest part of the conduits should be at least 4 feet below ground-level, and that no buildings should appear above ground-level, also that the outlets should not be visible from the Richmond bank of the river, and that the trees and shrubs should be protected as far as possible, any damage caused to them being made good by replanting.

A scheme to use a single tunnel of 14 feet 4 inches internal diameter, with numerous small outlets to the river, spaced over the full length of the Ait, was investigated. The difficulty at once arose that there would be considerable head-losses over this length and that the upstream outlets would therefore discharge a greater volume of effluent than those farther downstream. Furthermore, difficult ground for tunnelling was anticipated in the neighbourhood of the river, and such a large tunnel was therefore to be avoided, if possible. The determining factor, however, was that the effluent-conduit is a vital link, and could never be put out of use for inspection and repair, unless constructed in duplicate. Two outfalls of 11 feet internal diameter were therefore adopted.

Effluent-Conduits.

If the effluent-conduits discharging the final effluent from the purification-works to the Thames had been constructed on the hydraulic gradient they would have been very shallow under house property and through the site of an old ballast-pit filled in with rubbish of all kinds, including even old motor-buses. Boreholes revealed that they would have been in water-logged ballast and would have had to be driven in compressed air, at greatly increased cost. It was possible to overcome these difficulties by lowering the tunnels deep into the clay so as to form inverted siphons. Moreover, there would be no interference with navigation, nor would there be risk of damage to permanent work by subsequent dredging operations. As the effluent would contain virtually no suspended matter, there was no objection to the use of inverted siphons, and this method was accordingly adopted. The conduits are shown in plan in Fig. 14, Plate 1.

The maximum required discharge was 575,000,000 gallons per day, which is the aggregate discharging capacity of the sewers arriving at Mogden, and the head lost in the adopted design is 2·2 feet between Mogden and the first shaft. High water of ordinary spring tides at Isleworth Ait is 15·5 O.D., and therefore the hydraulic level at the down-shafts at Mogden at times of maximum flow and high water of ordinary spring tides is 17·7 O.D. The outlet-weirs of the storm-

water tanks are set at 20·0 O.D., and their copings at 24·0 O.D.; thus, should the highest flood-level recorded at Isleworth Ait occur at a time coincident with the maximum discharge, it would be necessary to shut off the flow from the purification-plant until the high tide dropped, and meanwhile to pass it through the storm-water tanks. It is, however, extremely unlikely that the maximum flow to Mogden will ever coincide with such extraordinary flood-conditions in the Thames; furthermore, the latter last for only half an hour or so.

During such conditions these tunnels are subject to an internal head of approximately 50 feet of water at the landward end. In view of the doubtful value of the earth surrounding a pressure-tunnel as a means of resisting internal pressure, it was decided to make the tunnels strong enough in themselves to withstand it. Fillets were therefore added to the longitudinal flanges of the cast-iron segments, to increase the strength to the required degree, and as an extra precaution the rings were all reeled during erection. The tunnels were graded down towards Mogden and connected there by a 4-foot 3-inch drainage-tunnel to the main sewage-pump-well, so that they can be emptied for inspection, cleaning and repair. The down-shafts connecting the effluent-culvert on the purification-works with the effluent-tunnels are built of cast-iron segments lined with concrete and brick, which was cheaper and more convenient than reinforced concrete.

To make the cast-iron segments thoroughly watertight, bituminous packing-rings were specified. Experience showed that these were best used in the form of "sandwiches," that is, with a wooden packing strip in between two bituminous strips. Plain bituminous packings may be satisfactory for hand-driven tunnels, but they squeeze out when uneven pressure of the rams of a shield is applied to correct variation of line or level. The "sandwich," however, overcame this disability and proved satisfactory.

Opinions vary as to the best way of lining a cast-iron segment tunnel with concrete and brickwork, and on different contracts different methods were permitted. Both of the principal methods were tried by both of the contractors engaged. It was specified that particular care should be taken to fill all hollow places with grout, on account of the internal pressure. The contractor was permitted to fill the key with brickwork in lieu of concrete if he wished, and one contractor elected first to do all the concreting and, later, as a second operation, to line the concrete with brickwork. He subsequently changed his method and laid the concrete and brickwork in the invert to springing-level first; then soffit brickwork was laid on ribs and laggings, concrete being packed behind as the bricks were

laid. The average volume of cement grout per linear foot of completed 11-foot tunnel required to fill all voids in concrete and brickwork inside the iron segments was measured, and was :—

(*a*) Whole arch concreted behind laggings (1,150 feet approx.) 1·14 bag of cement.

(*b*) Concrete and brickwork built together from springing to key (4,700 feet approx.) 0·9 bag of cement.

The first method has an advantage in that inspection and sounding of the concrete lining may be made before the brickwork has been started.

Effluent-Outfalls.

Each outfall passes through an intermediate "main shaft" on the island, and terminates in another "main shaft" about 100 yards farther on (Fig. 14, Plate 1). There are thus four main shafts on the Ait, spaced 100 yards apart, and from each main shaft, three 6-foot outlet-tunnels are driven out under the river, each outlet-tunnel terminating in a vertical shaft sunk from the bed of the river. The twelve outlets are located between 30 and 40 feet from low-water mark, the minimum depth of water over them being about 4 feet.

When both tunnels are discharging, the velocity at maximum flow will be 5·6 feet per second in the 11-foot tunnels, and 3·12 feet per second in each outlet-tunnel, so that the velocity-head produced is negligible. The hydraulic conditions of the general arrangement of outfalls are such that at maximum discharge, No. 1 outlet will pass only 7½ per cent. more than No. 12 outlet. The angle between the reaches upstream and downstream of the Ait is about 90 degrees, resulting in a rolling action of the current, which will assist diffusion. It was necessary to have the outlets below low-water mark, but the Port of London Authority would not permit even temporary works to encroach seriously on the width of the navigable channel, so that it was agreed to sink the outlet-shafts approximately on the line of the old low-water mark, and to widen the channel by dredging the bed and excavating the bank. Low-water mark has been set back an average distance of 12 yards, and the channel has been improved by sweeping in the new bank, parallel to the Richmond bank throughout.

As it was essential for the safety of the men employed, and to avoid any delay due to accidental flooding during construction, the tunnels were shield-driven, and air-locks and compressed-air plant were provided in case of need. As the cost of tunnelling is very little affected by the depth below surface, and since, in any event, the effluent-conduit must operate as an inverted siphon, it

was decided to fix the invert-levels of the main tunnels so as to ensure ample clay cover throughout. Bore-holes on the Ait and in the river-bed showed the minimum clay cover to be 12 feet, under the middle of the backwater.

A working site was provided for the contractor on the Middlesex bank of the river, with road-access and wharfage. The compressed-air plant was located on this site, out of reach of the highest recorded flood. A working-shaft was sunk on this site, and driving was commenced on No. 2 tunnel. The spoil was used to raise the Ait above the level of high water of ordinary spring tides (from 12 O.D. to 16 O.D.), and a bridge with an opening span was built on timber piles to afford access to the Ait at all states of the tide.

A short backshunt was driven out of the working-shaft towards Mogden, without a shield. The contractors on the adjoining section drove up to this backshunt, but were not permitted to use the working-shaft. No. 2 tunnel was driven through to the terminal shaft on the Ait, and there the shield was taken out and rebuilt in the terminal shaft of No. 1 tunnel. Driving then proceeded in the reverse direction to a point adjacent to the working-shaft on the bank, and a short connecting heading was driven from the working-shaft to No. 1 tunnel, for the purpose of bringing in materials. The shield was then dismantled, leaving the skin in position, and the shield employed under the other contract was then driven up to it to connect; it also was dismantled and the construction was completed inside the two skins. Before completion of the work the side connecting-heading was filled with concrete and the cast-iron lining was erected in the working shaft and surrounded with mass-concrete, the shaft being afterwards refilled to the surface with earth.

The intermediate shafts on the Ait were sunk from the surface on to the top of the tunnel, and the cast-iron-segment lining was removed as far as necessary. Special puddle-flanged segments were then built on, and embodied in the concrete walls of the shaft. Excavation was carried out inside steel sheet-piling 30 feet long, driven through the overlying gravel, and toed 12 feet into the blue clay. This was deeper than was necessary to ensure a water-seal, but an extra frame-setting in the timbering below was thus obviated.

The internal diameter of the shafts is 23 feet in order to accommodate the three 6-foot penstock-valves controlling the outlet-tunnels. It was thought that the outlet-tunnels would have to be driven in compressed air, and therefore a special steel air-chamber, with a vertical air-lock, was designed to enclose the "eyes" for the three outlet-tunnels from each shaft, while the lining of the main tunnel was proceeding in free air. Anchor-plates were built in to the floor

and sides of the shaft to accommodate this chamber, which was designed for the erection and starting-off of the small shield inside it. However, the location of the tunnels at a low level with ample clay cover was fully justified, because there was no need at any time to employ compressed air in any of the tunnels.

For each outlet (Fig. 15, Plate 1), a steel caisson 21 feet 6 inches in diameter was pitched in the river, and sunk by kentledge about 4 feet into the clay. Inside this caisson, in the dry, a steel sheet-pile box was driven, the side facing the bank being driven short to allow the shield to enter. Clay inside the steel sheet-piling was excavated, the floor sealed with 1 foot of concrete, and the shield driven into the pit, dismantled, and re-used on the next tunnel. Special puddle-flanged rings were then built on, the steel sheet-piling being left in place to act as outside formwork for the mass-concrete vertical outlet-shaft. Steel formwork was used for the inside of the vertical shaft. The outlet is bell-mouthed and is covered with a heavy wrought-iron grating with 6-inch clear openings.

Each outlet is controlled by a hand-operated 6-foot penstock valve bolted to the end of the cast-iron tunnel lining. The penstocks are single-faced only, and designed to resist water-pressure from the river with a head of 48 feet above invert-level. They were in place for several months with the full head of water on the river face, with the main tunnel empty, and proved to be perfectly watertight. Should silt accumulate and prevent the penstocks from closing, the seatings can be scoured by the operation of 6-inch sluice-valves.

There will always be an upward flow of effluent in the vertical outlet-shafts to prevent the entry of silt from the river-bed, and any outlet can be cleaned by closing other outlets at a suitable state of the tide and generating a high velocity of flow in it. During construction, as each outlet was completed a temporary wooden cover was fitted over the grating, but on bringing the outlets into use, it was discovered that one cover had become loose and had floated away, the outlet being filled almost solid with sand and gravel disturbed during dredging operations. By filling the main tunnel and opening the penstock-valve when the river-level was 12 feet lower, the obstruction was successfully removed.

Erosion of the banks of the Ait has been proceeding at the rate of about 1 foot per annum for at least 20 years; after excavation to meet the requirements of the Port of London Authority, therefore, the toe was held by piles, walings, and sheeters. The bank was then trimmed and pitched with Kentish rag-stone, the top 3 feet of pitching being grouted in cement mortar, and finished with a coping cast in situ.

Mogden Purification-Works.

Site and Construction-Programme.

A pumping-station and sewage-works belonging to the Heston and Isleworth Corporation (then the Urban District Council), together with orchards, market gardens, and allotments, occupied the site of the new purification-works. An area of 100 acres was acquired by the County Council under authority of the Act, together with as much more of the surrounding land as could be bought at reasonable prices, so that it now owns about 150 acres at Mogden. There was no need for the extra land so far as construction of works was concerned, but it was believed to be prudent to have the right to control development in the immediate vicinity of the works.

Although there was a visible fall across the land from Hall road on the west, the general configuration of the 100-acre site was flat, the level being roughly 32 to 35 O.D. Consideration of the hydraulics of discharge from the high-level sewer, through the new works and outfalls, to the Thames, and of the level of high tide, necessitated a final water-level at the works of 18 to 20 O.D. This difference in elevation of 15 feet necessitated the excavation of practically the whole site to an average depth of from 12 to 15 feet before the ordinary excavation for construction could be undertaken. About forty boreholes were put down at various points on the site, and it was established that most of this bulk excavation would be in water-bearing ballast, and that not only the foundation but the excavation for permanent construction-work would be in London clay.

It was obvious from the start that speedy design and construction would be seriously threatened by the presence on the site of the Heston and Isleworth sewage-works and their large area of sludge-lagoons, by the fact that only one outfall was available, and by the existence of the Duke of Northumberland's river and of a public road, Oak lane, subsequently closed, which cut across the site.

Because of the great quantity of material to be excavated and of the large cost of removing it from the site of works for disposal elsewhere in an already developed district, it was economic to dispose of all surplus by forming embankments around the site to screen the works from surrounding property. Even so, the quantity to be dealt with was such that the area remaining available for the works would be substantially reduced if all the surplus were disposed of in this manner. Fortunately, the water-bearing ballast proved to be a marketable commodity. A contract for ballast excavation down to the clay level over a restricted area was therefore prepared

immediately, in order that this work could be carried out while the constructional work was being designed.

At that time the intention of the County Council was to cater for a population of 750,000 persons, only about one-third of the ultimate population. Plant representing one-third of the ultimate lay-out was therefore contemplated, and it was found that with care such works could be located to avoid the old Heston and Isleworth sewage-works, leaving the latter to perform their duty until the new plant was in operation, ready to supersede them and to receive the additional flow from the twenty-six other works. The old Heston and Isleworth works would then have been demolished and the site made available for future extensions to the new plant. The ballast-excavation contract was therefore originally drawn up to lower the general ground-level over an area of about 20 acres located inside the future embankments. Conditions of contract were prepared, not only to allow of as wide a choice of excavation-methods as possible, but to encourage the tendering of ballast-merchants. The probability of excavation by pumping was recognized. It was laid down that the successful tenderer was at liberty to dump the ballast on the western part of the site, and he was to provide washing, grading, and loading plant there in order that the Council could immediately start disposing of their ballast to the contractors engaged on the sewers, and, if possible, to the public. At the same time, it was made clear that the Council wished to encourage alternative suggestions for disposal of the ballast. This contract, called "M.1," was eventually awarded to the Ham River Grit Company, Ltd., who, under an alternative offer, assumed the ownership of the ballast and became responsible for its removal. If it had been known at that time that purification-works for 1,250,000 persons—an increase of 66 per cent.—would be required in 1935, the Council would not have entered into a contract in this form, and the contractors would not have been empowered to occupy the allotted site for dumping ballast, which caused so much congestion to constructional operations at a later stage.

The overburden of top soil was first removed by excavators and by scrapers drawn by caterpillar-tractor; later, two pontoons carrying 8-inch pumps were used to deliver the ballast into five barges which were towed by motor-boats to the wharf, where the ballast was again sucked out by a 12-inch pump, and delivered to a storage-hopper of 200 tons capacity. Two 6-inch pumps supplied water to the barges, both for washing and to facilitate the pumping. From the hopper an aerial ropeway some 600 yards in length and of a carrying capacity of 250 tons per hour delivered the ballast to the dump, some of it being diverted on the way to the screening, washing

and crushing plant for immediate treatment and sale. A method of spreading the material in the dump by using a scraper hauled by wire ropes was amplified by a flume system, whereby a powerful pump drawing water from the Duke of Northumberland's river washed the material away from the ropeway. At its maximum the ballast-dump occupied an area of about 17 acres, rising to about 40 feet in height at the ropeway end.

While this work was in progress, and the design of the works was proceeding, the development of the County was causing uneasiness owing to its increasing rapidity. When the 1931 census-returns became known they showed the population to have grown to 711,263 (21,263 more than the works were being designed to serve in 1935). It then appeared that a population approaching 900,000 would be reached in 1935, and plans had to be altered at once so that one-half of the ultimate works contemplated would have to be constructed instead of one-third, so as to allow for a population of 1,000,000 instead of 750,000. This was a most serious setback to the chances of completion by October, 1935, the stipulated date, because of the complications and extra work to be done on the site. Had this necessity not arisen, the ultimate lay-out would have consisted of three main batteries of purification-plant, with the Duke's river diverted to run between the second and third batteries, and the initial construction would have consisted of the first battery only, without any interference with the Heston and Isleworth works or the Duke's river. When it appeared that half the ultimate works would have to be built, the lay-out had to be altered. A great deal of the work already carried out up to that time on the design of the works had therefore to be scrapped late in 1931, the new lay-out introducing new features which can only be regarded as some of the major impediments to the completion of the work by October, 1935. The chief additional difficulties were the demolition of the old Heston and Isleworth sewage-works, pumping-station, and outfall, the severance of the site caused by the diversion of the river at an elevation of about 12 feet above the new level of the purification-works, and the acute congestion of the site during construction.

Except for the temporary retention of the pumping-station and certain other necessary plant, the old works were demolished at the earliest moment and temporary cast-iron pipe-lines were laid to carry the sewage for treatment in the first of the new storm-water tanks to be completed. From there it had to be lifted by temporary pumping-plant installed in the new main effluent-culvert, through a temporary steel rising main about ½ mile long into the old outfall. The sludge was pumped back across the site through another temporary main to digestion-lagoons constructed with earth em-

bankments on the north-west corner of the site. It was not always possible on such a congested site to locate temporary pipe-lines so that they were clear even of permanent work, and further diversions were inevitable. The use and maintenance of such temporary plant in co-ordination with the contractors' operations required considerable ingenuity.

Such difficulties were aggravated still more in 1933, when the population-estimates of the constituent districts showed such a substantial increase that the Council again had to face the possibility of the works then being constructed on the basis of 1,000,000 people being only just sufficient for the requirements anticipated to arise in 1935. Again an effort had to be made to crowd still more work into the available working time, and on this occasion the means employed were to make provision for aeration-tanks and final-settling-tanks sufficient to deal with a population of 1,250,000. This was made possible by incorporating into the advertised conditions of the appropriate contract a provision whereby, whilst the contractor was bound to complete the work originally contemplated by the contract date of completion, he was encouraged in addition to do as much supplemental work as he could in the same time, payment for such extra work being fixed at rates giving a 10-per-cent. bonus on those tendered for the main contract.

It was anticipated that the pumping-dry of the site, after the process of ballast-getting, would be a big undertaking involving the lowering of ground-water over some considerable area in the district. The ballast-pit covered about 23 acres, and contained about 45 million gallons. The rate of inflow was surprisingly low, averaging only about 1 million gallons per week over a period of about 4 months. The Council installed a small temporary pumping-plant to keep the site drained, and operated it throughout the period of construction.

During the progress of the ballast-getting all the water used in washing the ballast was returned to the pool at the south end, leaving a deposit of very fine silt which attained a much greater volume than was expected. It covered many acres, and was so soft that it was impossible to explore. Its drainage to a dry state fit for excavation became a serious problem, because of its retentiveness of water, and though weeds began to grow on a fairly dry crust the bulk beneath was still in a fluid state. In spite of drainage-channels cut with considerable difficulty from the site-drainage pumping-station, it would not readily drain even in dry hot weather, and the bulk of it had ultimately to be dealt with in a semi-fluid state, causing difficulty and some delay.

Construction work was divided into four main contracts. In the circumstances of limited site-area, the ideal would have been to let

one large contract for the whole of the work and to hand over the complete site to the one contractor, but it must be remembered that the design period was included in the time-limit of 4 years allowed, and the preparation of the contract design in all its multitudinous details was in itself such a large undertaking that punctual completion of the entire scheme was only possible by proceeding with design and construction simultaneously. The detailed plans of the storm-water tanks were therefore rushed forward, and contract "M.2" was let in March, 1933, for their construction, whilst designs for the secondary sedimentation-tanks, aeration-tanks, operating-gallery, final separating-tanks, return-sludge pumping-station, river-diversion, and public footpath were prepared and let as contract "M.3" in October, 1933, together with the supplemental work of additional aeration-tanks, operating-gallery, and final separating-tanks. Similarly, contract "M.4" for pumping-station, workshops, screen-house, grit-channels, primary sedimentation-tanks, raw-sludge pumping-station, and administration-building was let in May, 1934, after contracts "M.2" and "M.3" were in full swing, thus taking advantage of the extra time available for their detailed design. Contract "M.5" followed in July, 1934, for the construction of compressor-house and sludge-digestion plant. In the case of every contract, particularly where machinery was involved, adjustment of detailed design proceeded throughout the construction-period.

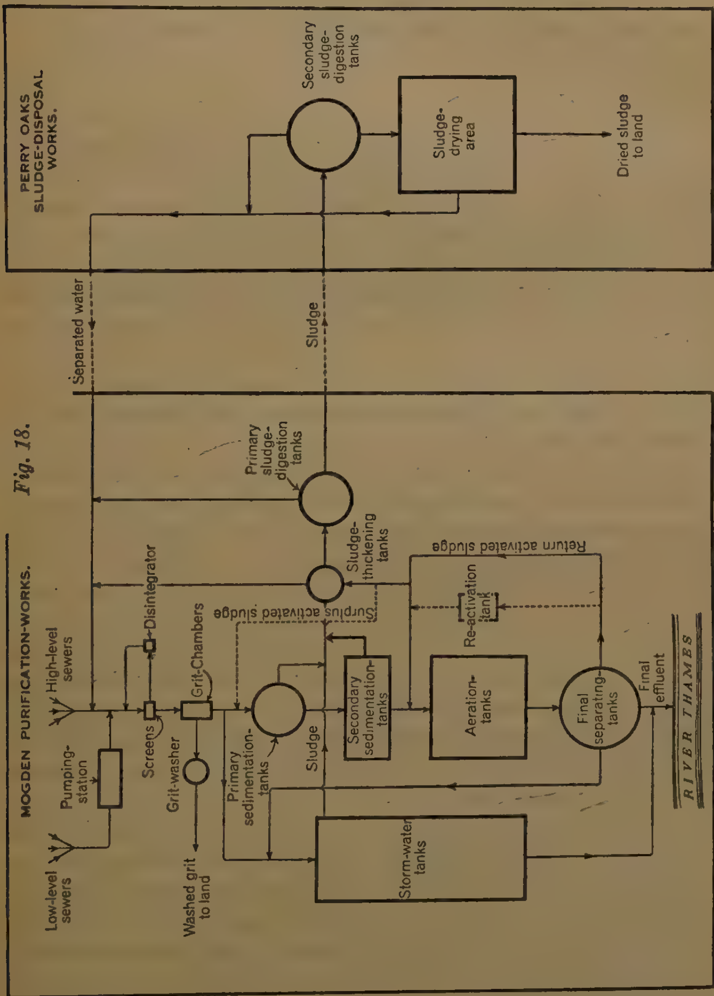
The lay-out finally adopted (Fig. 16, Plate 2, and *Fig. 17*, facing p. 476) embodies an avenue of trees 1,600 feet in length on the centre-line of the ultimate works, running from the main entrance at the south to the administration-building at the north. In this avenue are the main works-road, the Duke's river, and a public footpath running side by side. With the exception of certain plant which will not require ultimate duplication, the plant constructed on the east of this central line is designed for a population of 1,000,000 persons.

Works provided.

For the purpose of producing the high standard of effluent required, the following component units were installed :—

Coarse and medium screens, with macerating plant.
Grit-collecting channels, dredging and washing plant.
Separating weir for storm-water in excess of three times the dry-weather flow.
Primary sedimentation-tanks.
Secondary sedimentation-tanks.
Aeration-tanks.
Final separating-tanks.
Storm-water tanks.
Sludge-digestion plant.

The method of connexion of the plant is as shown in *Fig. 18.* Certain units, being uneconomical to construct in modified form for enlargement at a later date, were built at once large enough to serve the ultimate population of 2,000,000. Although the drainage-



METHOD OF CONNEXION OF MOGDEN AND PERRY OAKS PLANTS.

district had reached approximately only one-half of its final development, it was considered possible that the wet-weather flow might reach the ultimate maximum theoretical flow even in the early years of the operation of the scheme, because the sewers were being constructed of suitable size and arrangements were being made

deliberately to take advantage of this capacity for flood-relief. Hence the main sewage pumping-station, screening-plant, grit-channels, and storm-water tanks were provided for the maximum duty, while the administration-building, power- and compressor-house, and main service building, being uneconomic to extend in sections, were constructed large enough for final requirements.

Screening.

The object of screening the sewage at the Mogden works is not merely the ordinary one of removing the screened material for disposal by dumping. It is necessary in order to protect sludge-removal mechanism in sedimentation-tanks, sludge-pumps, and other mechanical plant freely employed on the works. The location of the screens above the grit-plant was decided by the determination to produce grit from the grit-channels in as clean a condition as possible.

Disintegration of screenings and their return to the sewage for settlement in sedimentation-tanks as sludge was adopted as the method of disposal. Dumping or incineration were ruled out as potential sources of smell-nuisance. Alternative methods of disposal of the disintegrated screenings would be by digestion in a separate tank, or by pumping direct to the sludge-digestion plant. With the maceration-plant adopted, a large volume of water is necessary for operation, and de-watering plant would therefore be necessary before digestion. Furthermore, the necessity for thorough mixing with the sludge to give uniformity of consistency would create other problems. By returning the disintegrated screenings to the sewage above the screens, complete disintegration is ensured, reasonable mixing with the solids to be brought down in the sedimentation-tanks is facilitated, and the cost of conveyance of the screenings from the screening plant is eliminated.

There are six grit-channels and therefore six approach-channels to them from the outfall-sewer. The screens are situated in the approach-channels, to afford greater ease and flexibility of control than could be afforded by one large screening-plant in a bay of the main outfall-sewer. Ample screen-area and space for operation of the screening plant is thus provided (*Fig. 19*, facing p. 477). Velocity-conditions over the range of flow are thereby improved and the settlement of grit in the approach-channels is reduced.

Protection for the raking gear of the medium screens and for the disintegrators is necessary; inclined screens with bars at 4-inch spaces to catch large solids, such as timber, were therefore installed just above the screens. For the small amount of material expected

to be arrested by these racks, power-driven equipment was considered unnecessary. Owing to the length of the racks, however, raking by hand was impracticable, and in the type chosen a compromise was effected by the provision of a manually-operated mechanical rake wound up and down by means of a winch, the screenings being raked off at the top by hand rake into a steel trough for inspection. The material unsuitable for maceration is removed for disposal, and the remaining material is dropped into the channel to be picked up on the medium screen, the greater proportion of screenings arrested here being dealt with by the disintegrators.

The bar screens which follow have ¾-inch openings, and their rakes are electrically operated by automatic gear. Air-pipes extending down into the sewage above and below the screens register differential air-pressure on a gauge, and are made to operate an electric relay at predetermined pressure and hence to control automatically the operation of the rakes.

Screenings are conveyed by belt conveyor to two receiving hoppers, and the disintegrators are fed from the bottom of the hopper, river-water or sewage being used as a carrier. Water-pipes are arranged with a two-way delivery valve by means of which the water to the disintegrator may be taken through the hopper, direct to the disintegrator, or both ways in any desired proportion. This enables the screenings to be fed to the disintegrator at any desired degree of dilution. Each disintegrator is driven by a variable-speed motor, affording the maximum flexibility in operation.

Grit-Channels.

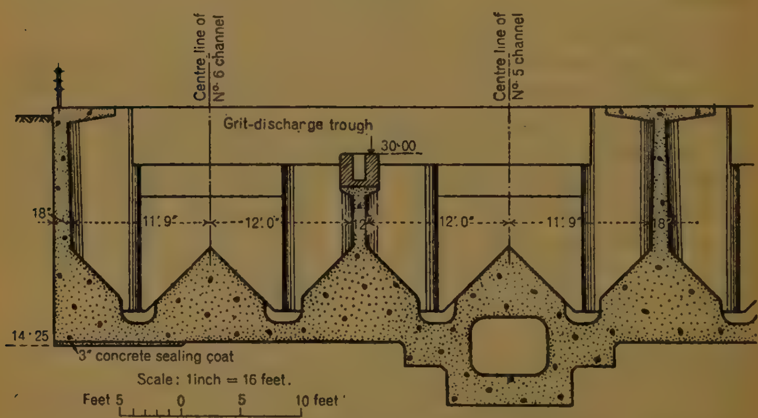
As it is conceivable that a volume equivalent to the maximum capacity of the sewers might be received in the early years, it was necessary to provide for a range of flow in the grit-channels varying from, say, one-half of dry-weather flow at the inception of the works (20,000,000 gallons per day) to the aggregate discharge-capacity of the sewers (575,000,000 gallons per day); in other words, a range of nearly 1 : 30.

Hitherto the main difficulty in design has been the maintenance of a uniform velocity of flow throughout the entire range of discharge under the greatly varying head-conditions, both up and down stream. Many previous attempts have been made to solve the problem by the use of proportional weirs and other devices, with varying degrees of success, but all such methods have been accompanied by serious drawbacks. The design evolved for the Mogden plant offers virtually a complete solution to a problem of outstanding complication in this particular case, involving not only that of the

grit-channel itself but also a question of gauging of more than ordinary difficulty.

The plant adopted relies on the provision of independent grit-channel units, each arranged in series, with a standing-wave flume (*Fig. 20*, facing p. 477) used for purposes of gauging the flow through the unit, but at the same time fulfilling the additional function of controlling the surface-level of the sewage upstream in the grit-channel. By making the cross-section of the grit-channel such that a curve plotted to show the cross-sectional area against the depth of flow exactly corresponds to the curve showing the discharge through the flume at the same depth, a uniform velocity of any predetermined magnitude is obtained. In the case under consideration, a velocity

Fig. 21.



GRIT-CHANNELS.

in the grit-channel of 1 foot per second was selected, and in consequence the curve of discharge on a base of cusecs must be made to coincide with the curve of channel cross-section on a base of square feet, both curves being plotted against water-depth. It can be shown that the required shape of channel section to fulfil this condition is parabolic. If a single channel were used for the whole range, its breadth would be 180 feet, to give a maximum depth of 9 feet and maximum flow of 1,065 cusecs. Six channels each 30 feet wide would answer the same purpose, but site-conditions dictated reduction to 24 feet wide each, and the section ultimately decided upon (*Fig. 21*) is a compromise between the parabolic theoretical section and a section of economic construction. Over the greater part of the range the areas are practically identical, the maximum

departure from the theoretical curve not exceeding 5 per cent. and occurring at the upper end of the range.

One great advantage of the arrangement adopted is that the flumes act as automatic head-balancers between the sewer-conditions and the downstream conditions of levels in the culverts leading to the purification-plant. If sufficient head is available and the flume is maintained under standing-wave conditions, the grit-channel control is absolutely independent of downstream water-levels. These conditions will ultimately be fulfilled, but in the early years, owing to the smaller head available, it is impossible to operate the flumes under "free-discharge" conditions when the flow is very low. The effect of drowning is to retard the velocity of flow in the grit-channels, but provided that the drowning is not extreme, the reduction in velocity is not very great, and this can be regarded as a temporary condition which will disappear as the population increases.

Within the limits of the drowning referred to, the velocity of 1 foot per second is maintained in the grit-channels irrespective of the number in operation. It is important, however, to regulate this operation in such a manner that proper velocities are maintained in the outfall-sewer above the works, the level in which is also indirectly controlled by the flumes. If the bringing into action of an additional unit with increasing flow is delayed unnecessarily, the flumes in operation will be controlling the smaller number of channels at too high a level from the point of view of the sewer, at the expense of the velocity in the sewer, and grit would be deposited in the sewer before reaching the works.

For removal of the deposited grit, dredging by pumping was selected as being cleaner and more economical than the bucket elevator or grab normally employed. Double handling is inevitable in both cases, but if the grit is being pumped, the washing action can be turned to good account. If the resultant grit is to be easy to dispose of, its cleanliness is important. For these reasons, the plant adopted consists of a centrifugal pump mounted on a travelling gantry which spans across two channels, the pump travelling along the gantry to lift grit from either. Three gantries are therefore required for the complete installation of six channels. The pumps deliver into transverse launders on the gantries, which in turn discharge into delivery-troughs built in the division walls, and communicating directly with vertical cylindrical settling-tanks in the screen-house. A mixture of grit and sewage flows upwards in these tanks at a rate of 0·2 foot per second and grit settles in the conical storage space at the bottom, the sewage passing over the peripheral weir at the top and returning to the grit-channels. After settlement of dredged material is complete, river-

water is pumped into the settling tank, and organic sludge thoroughly washed out to leave only clean grit, the wash-water passing over the weir into the grit-channel. The grit is then conveyed to a disposal-area by pumping, using river-water as a carrier. Water drains from the clean grit and ultimately returns to the river as subsoil water. No immediate use for this grit has yet been found, but utilization in the future is a possibility.

Each grit-channel has two penstocks, one above it and one below it, so that the closing of any pair of penstocks completely isolates a channel with its screen and flume. As the flow varies, the penstocks operate automatically in pairs, opening or closing the channels one after another. It is particularly important that these penstocks should be reliable in operation, because failure to open at the appropriate level of sewage in the sewer might cause serious surcharge. Hydraulically-operated penstocks were chosen for this reason, the hydraulic accumulator having sufficient capacity to operate all twelve penstocks without recharging. Even a stoppage of electric supply would not, therefore, affect the plant, the necessary power for operation being always available. If manual opening should ever be necessary, hand-pumping of an hydraulic penstock will have the advantage of higher efficiency than hand-operation of an electrical penstock, and the former can therefore be opened in a shorter time. The penstocks are automatically controlled by a 4-foot diameter float actuated by the sewage in the sewer, and operating hydraulic relays at predetermined levels. There is a time-lag between the opening of the upstream and the downstream penstock of any one pair, to allow the grit-channel to fill up to the appropriate level. The control plant is arranged to give as much flexibility in operation as possible. Penstocks can be operated at any level of sewage, in any order, and the time of opening and closing can be regulated over a wide range, the minimum being $1\frac{1}{2}$ minute.

In the event of failure of the control gear at times of high flow in the sewers, a siphon spillway, situated immediately above the upstream penstocks, will come into operation if the sewage-level in the outfall sewer reaches danger-mark, and the discharge will by-pass the grit-channels to a point on the downstream side at a maximum rate equal to rather more than the maximum discharging capacity of one grit-channel.

Gauging.

Since some of the sewage arriving at the works comes from the high-level sewer, and some from the low-level sewers, and the gauging of the low-level volumes is easily effected by a venturi-meter on each pump-delivery, the simplest way of arriving at the

flow of the high-level sewer is by subtraction of the aggregate pumping rates at any given moment from the total flow leaving the grit-channels. Flows to the primary sedimentation-tanks to receive full purification are gauged by the main venturi-meter situated below the separating weir, and just below it is a control-penstock operated by the meter and limiting the flow to the purification-plant to rates not exceeding three times the dry-weather flow. This penstock is readily adjustable to suit the current conditions of population, and the excess flows which cannot pass it overflow the separating weir and discharge to the storm-water tanks. All meter readings are transmitted electrically to an instrument-board consisting of five panels in the main pumping-station, each panel having an indicator, integrator, and recorder. The panel shows the electrically-summated flume-readings of the total flow through the grit-channels, including the low-level flow (pumping-station meters, summated electrically), the high-level flow (obtained by mechanical interconnexion of meters), the storm-water flow (obtained by mechanical interconnexion of meters), and the flow to the purification-works (main venturi-meter reading). The meters operated by the venturi-tubes on the pumping-mains are mercury manometers; the main meter depends on water-columns with displacers, and the upstream and throat levels of the flumes are obtained by floats.

An interesting feature of the meters is a new method of electrical summation giving an improved degree of accuracy at low flows, and enabling the indicator and recorder scales to be opened out at the lower end of the range. Thus the normal working range up to, say, twice the dry-weather flow covers a considerable portion of the scale, and, in the case of the recorder, the chart-width is used to the best advantage. The electrical transmission of meter-readings operates on an a.c. supply, for which duplicate motor-generator sets are used, one as a standby, and the recorder drums can then be synchronized by the use of a.c. synchronous motors.

Schedule of Main Plant in Screen-House and at Grit-Channels.

Six coarse screens, 3-inch by 1-inch bars, 4-inch spaces.

Six medium screens, 3-inch by ½-inch bars tapering to ¼-inch, with ⅜-inch spaces; mechanical rake cleaned by revolving brushes; operated by 7½-HP. electric motor.

Three belt conveyors, 18 inches wide, 43 feet 6 inches long; speed 100 feet per minute. Driven by 1½-HP. electric motors (reversible).

Two disintegrators, vertical-spindle, 12-inch; approximate capacity 400–450 cubic feet of screenings each per hour; water-consumption 1,000 gallons per minute; maximum static head about 13 feet; driven by 15-HP. variable-speed electric motors.

Automatic screen-control gear. Differential head registered on gauge, operating switchgear.

Three grit-dredgers, 6-inch fullway pumps delivering 1,000 gallons per minute each, driven by 20-HP. variable-speed motors; electric exhauster for priming; hand-operated swivelling suction-pipe; hand-operated travelling carriage.

Three gantries, 46-feet span; electrically driven; speed 2 to 20 feet per minute.

Two grit-disposal 5-inch fullway pumps, delivering 700 gallons per minute each; driven by 50-HP. variable-speed motors; the same pumps can be used for draining the grit-channels, with separate delivery to discharge to main sewer.

Two water-pumps (for disintegrators, grit-disposal and washing); 7-inch pumps delivering 1,200 gallons per minute each; driven by 30-HP. variable-speed motors.

Grit-channel control penstocks, inlet 4 feet 6 inches by 9 feet, outlet 6 feet by 8 feet, hydraulically operated; speed of opening adjustable, minimum time 1½ minute; operated automatically by float-controlled hydraulic relays; outlet-penstock commences to open when inlet-penstock reaches top of travel.

Hydraulic accumulator (air and oil accumulator). Working pressure 400–500 lbs. per square inch; capacity sufficient for opening twelve penstocks without recharging; automatic charging by two electrically-driven three-throw oil-pumps with emergency hand-operated pump; alarm device operating if pressure drops below recharging value.

Meters: In the pumping-station. Six 15-inch and six 30-inch venturi-meters on pump delivery-pipes.

At the grit-channels. Six standing-wave flume-meters with double floats for "drowned" conditions; throat width 2 feet 2¼ inches.

Flow to purification-works. Main venturi-meter on approach-culvert, 5 feet by 6 feet 6 inches.

Electrical transmission, summation, indication, integration and recording on all meters.

Primary and Secondary Sedimentation-Tanks.

There is a wide range of opinion as to the time necessary for sedimentation prior to complete or partial purification in activated-sludge plants. A 12-hour detention-period is not uncommon in Great Britain, whereas in America and Germany 1 or 2 hours are considered sufficient. Although it may not be reasonable to expect the activated-sludge process to deal with all the sludge, too long a sedimentation-period may produce septic sewage, with ill effects on the process which follows. The total detention-period for the sedimentation-tanks was fixed at 8 hours, and it is believed that this will prove sufficient, more particularly because it is expected that some of the units will be shut down in long periods of dry-weather flow, and it will be a matter for experiment to find out if a much shorter detention-period will not be sufficient. Prompt removal of sludge from sedimentation-tanks is a feature of modern practice, and since the bulk of sludge is settled at an early stage in the process, it becomes necessary to adopt means of removing the bulk of this coarser sludge rapidly and at frequent intervals. For this reason, amongst others, two-stage sedimentation was provided. Of the

Fig. 22.

PRIMARY SEDIMENTATION-TANKS.

Fig. 25.

INLET END OF SECONDARY SEDIMENTATION-TANK, SHOWING SLUDGE-REMOVAL MECHANISM.

5120.

Fig. 29.

AERATION-TANKS: OPERATING-GALLERY AND CHANNELS.

Fig. 31.

STORM-WATER TANK, SHOWING INLET END.

total detention-period of 8 hours approximately one-third is allowed for primary and two-thirds for secondary sedimentation.

Since the bulk of the sludge settles in the primary sedimentation-tanks, it is necessary to remove it more frequently than the thinner sludge from the secondary sedimentation-tanks. Arrangements have also been made to dispose of the surplus activated sludge by pumping it into the inlet-culverts of the primary tanks, and this renders the rapid removal of sludge still more necessary to prevent septic action. For both these reasons frequent sludge-removal was essential, and machines were selected suitable for continuous operation if found necessary.

The secondary sedimentation-tanks, being twice the capacity of the primary tanks, and dealing with lighter sludge, are provided with the Mieder type of machine, which, whilst cheaper, is able to remove sludge sufficiently frequently for the purpose in view.

Primary Sedimentation-Tanks.

Eight circular units are provided (*Fig. 22*, facing p. 510) of a total capacity of 4,500,000 gallons, allowing a detention-period of 2·66 hours for a dry-weather flow of 40,000,000 gallons per day, assumed for a population of 1,000,000, the eight units giving ample scope for flexibility in working. The tanks are each 90 feet in diameter and 11 feet deep (Fig. 23, Plate 2); they are provided with central inlets and peripheral weirs, and are arranged in two groups of four units each, space being left between the groups for auxiliary plant such as pre-aeration or grease-removal tanks, should future conditions require it. The inlet-culvert originates at the south end of the separating-weir chamber and, after passing the main control-penstock, divides into two culverts running east and west to the centre of each group, at which point a 40-inch feed-pipe to each tank leaves vertically from the bottom of the culvert floor. Each feed-pipe, controlled by a 40-inch sluice-valve, ends at the centre of the tank in a vertical bellmouth 3 feet below the water-surface. The inlet-culverts are each 8 feet wide and 4 feet high, and for part of their length are incorporated in one structure with the outlet-culverts, 4 feet wide by 8 feet high, superimposed on them. The general layout of supply-culverts and mains has been made perfectly symmetrical in order to ensure equal distribution of flow over the entire battery.

Effluent flowing over the peripheral weir passes round a circular channel and enters the outlet-culvert through 5-foot by 4-foot penstocks, which can be operated by portable electric drive. The peripheral weir is situated on the outside of the main tank wall, which acts as a scum-baffle. The floors of the tanks are slightly conical, being laid to a gradient of 1 in 10.

The sludge-removal mechanism is of the "Rota" type, consisting of a rotating bridge of 96 feet 6 inches span, from which are suspended eleven rubber-edged scraper blades. Each blade sweeps an annular ring, and being freely hung takes up its own sweeping position and is able to ride over any obstruction on the floor of the tank. The rotating bridge is mounted on two end carriages, one wheel of each carriage being driven by a 5-HP. motor mounted at the middle of the bridge. Current is supplied to the motor by a cable laid through a 3-inch pipe in the floor of the tank, and carried up the side of the inlet-pipe to slip-rings.

From the centre of the rotating bridge is suspended a diffusion-box, 14 feet in diameter and 6 feet deep, constructed of copper-bearing steel, and projecting 6 inches above water-level. The skimming device consists of a fixed blade at a slight angle to the bridge and a movable blade operated by means of the engagement of a toothed wheel on vertical teeth fixed on the coping of the tank. The movable blade swings towards the scum floating arm, which is depressed by means of a skid attached to the rotating bridge. The scum trapped by the swinging blade is passed down the scum-pipe, accompanied by a quantity of tank liquor which is adjusted by an alteration of the skid position. Sludge is swept to a well in the centre of the tanks, whence it is drawn off under hydrostatic head through a 12-inch sludge-pipe provided with a visible outlet into a sludge-hopper for inspection when necessary. It is possible, however, to draw off sludge directly by the sludge-pumps. The tanks can also be emptied by this means, and, in the event of a block in the suction-mains, pressure can be applied from the pumps to clear it. Scum is collected by means of the sludge-removal machine, and passes through a floating arm into the sludge-inspection chamber.

Secondary Sedimentation-Tanks.

Any sudden variation in the strength of the crude sewage will be largely diffused in the primary tanks, but the secondary tanks carry the equalizing of strength a stage further, as well as arresting much of the lighter suspended matter. Four tanks are provided having an aggregate capacity of 9,000,000 gallons, which is equivalent to a detention-period of 5·4 hours on a dry-weather flow of 40,000,000 gallons per day. Each is approximately 200 feet long by 150 feet wide with an average depth of 12 feet, with a central wall with large ports in it running longitudinally down the middle to carry a rail for the sludge-removal mechanism, thus dividing each tank into two bays of approximately 75 feet wide by 200 feet long.

Flow to the tanks is controlled by four penstocks in one chamber,

two of the penstocks being 6 feet 9 inches by 5 feet 6 inches and two 5 feet by 5 feet 6 inches. A supply-culvert connects each penstock to one of the four tanks, and as these culverts are not all of the same length, the cross-sections are so designed as to give identical total hydraulic losses in order to ensure equal distribution throughout the battery. At the end of each of the supply-culverts the flow is directed into two 48-inch cast-iron feed-pipes, from which four 36-inch pipes feed each tank, discharging to the bottom of a diffusing channel 3 feet wide (Fig. 24, Plate 2). Equalized distribution is effected by means of a weir running the full width of the tank and forming one side of the diffusing-channel. Sludge-deposit at this point is obviated by concrete filling, carefully shaped to avoid quiescent conditions. Concrete baffles 2 inches thick are supported from precast concrete brackets built into the concrete weir wall, and give the sewage an initial downward velocity towards the sludge-hoppers. Eight sludge-hoppers are provided per tank, 18 feet 9 inches square on plan, the depth from tank floor to the bottom of the hopper being 14 feet. Sludge is discharged under hydrostatic head from the bottom of each hopper to a sludge-inspection chamber, of which there are two to each tank. Arrangements are made, as in the case of the primary tanks, for visible sludge-discharge where desired or for removal by pump suction. At the outlet end of the tanks, a platform forms the scum-baffle. Scum collected against this baffle by the sludge-removal mechanism is swept across the tanks by hand, and passed through weir penstocks, together with some tank-liquor, into a pipe discharging to the raw-sludge pumping-station. Tank-effluent discharges over a weir into an outlet-culvert 9 feet wide located partly under the curved end wall of the tanks. This curved end, with a radius of 22 feet 2½ inches, is shaped to fit the sweep of the blade of the sludge-scraper when it is being lowered after the skimming operation into the sludging position. The sludge-removal mechanism (*Fig. 25*, facing p. 510) is of the Mieder type and is very similar in construction to the machines adopted for the storm-water tanks (p. 521). The six hoists for the sweeping blade are operated by a 3-HP. motor. The floor of the tank is horizontal in cross-section, but has a longitudinal fall of 12 inches towards the sludge-hoppers at the inlet end. The normal velocities of the sweeping blade are 8 feet per minute down the tank for scum-removal, and 4 feet per minute back again for sludge-collection.

One of the merits of these scrapers is that any number of tanks can be served, a transporter-carriage travelling on a transverse track being provided for this purpose as an integral part of the design of the battery.

Aeration-Tanks.

Since the standard of purity of effluent demanded by the Act is (*a*) not more than 2 parts of dissolved oxygen to be taken up per 100,000 parts of effluent in 5 days at a temperature of 65° F., and (*b*) not more than 3 parts per 100,000 of suspended matter to be contained, the secondary or biological purification had to be taken towards completion.

Quite apart from the lack of available area of land and of hydraulic head, which in themselves had always ruled out the possibility of older processes such as percolating-filters, the absolute necessity to avoid all risks of fly and smell nuisance in a well-developed district was a deciding factor in the choice of the activated-sludge process. The progress made in purification-processes in the last 40 years has been so enormous that, roughly speaking, 1 acre of activated-sludge plant will do the same work as 10 acres of percolating-filters, 20 acres of contact-beds, or not less than 500 acres of land-treatment. The advantages claimed for purification by the activated-sludge treatment may be summarized as follows :—

(i) freedom from fly and smell nuisance ;
(ii) economy in land utilized ;
(iii) low hydraulic head required ;
(iv) suitability to large works with skilled scientific management ;
(v) economy in first cost ;
(vi) economy in operation, particularly by the utilization of power generated from methane gas evolved by sludge-digestion.

The basis of design finally adopted allowed for a 12-hour detention-period (based on dry-weather flow) inclusive of sludge-reconditioning units, but exclusive of the miscellaneous aerated connecting-channels, etc., and making no allowance for the volume of return sludge. That part of the plant which lies east of the main access road is designed for a population of 1,000,000 persons, and the plant west of the river —the " supplemental work "—for a population of 250,000 persons.

Supervision and control of such plant may be made needlessly difficult and costly by failure to concentrate all the meters and controls in a convenient manner, and the inlets and outlets to the various units of the aeration-tanks have therefore been arranged all at the south end, where the operating-gallery has been designed to form the control-platform for the complete plant. Considerable advantages from other aspects are gained by this form of layout. Complete symmetry of unit construction is attained, greatly simplifying the provision of expansion-joints in the concrete structure

and rendering the system of compressed-air pipes simple and straightforward. To supply the main plant east of the river, the flow from the penstock-chamber below the secondary sedimentation-tanks is divided into two channels running from north to south between the two batteries of units. At the southern end of the aeration-tanks, they turn east and west respectively, being thence incorporated in the operating-gallery construction, together with the sludge-return channel and the mixed-liquor channel.

At the point of turning, the channels are contiguous to the sludge-return pumping-station, whence a predetermined volume of activated sludge is discharged into them. Up to this point no diffused air is used, although the channels from the secondary sedimentation-tanks are so constructed that they may readily be made to afford pre-aeration to the tank liquor if required. Each battery of aeration-tanks consists of six units, each unit (Fig. 26, Plate 2) comprising four channels 400 feet long, 15 feet wide, and 12 feet deep from nominal water-level to the top of the diffusers, and with a freeboard of 2 feet. The flow to each battery is gauged by passing through a 6-foot by 5-foot rectangular venturi-meter, immediately upstream of the return-sludge discharge. After receiving the activated sludge returned by the pumps, however, the channels are aerated, and the mixture discharged at intervals, through electrically-operated penstocks 6 feet high by 4 feet wide, to the various units of the aeration-tanks, in which the sewage travels up and down the four channels through a total distance of 1,600 feet. The purified sewage leaves the aeration-channels through 36-inch venturi-tubes built through the operating-gallery into the mixed-liquor channel, by means of which it is distributed to the final separating-tanks. Here the activated sludge is separated from the purified effluent, which is discharged to the river, whilst the sludge is delivered to the sludge-return channel in the operating-gallery, to be dealt with by the pumps in the return-sludge pumping-station either for return to the incoming tank-effluent or as surplus.

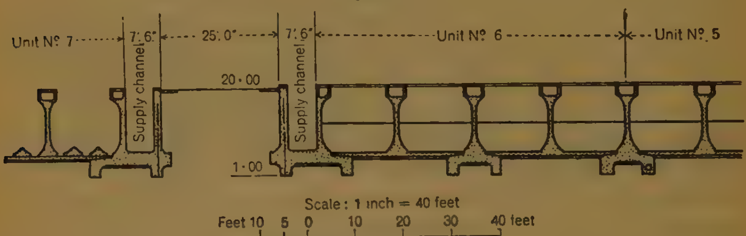
Two units of each battery can be used for re-activation of return-sludge. The western battery is arranged on the "longitudinal ridge and furrow" principle (*Fig. 27*, p. 516), and has three rows of 6-inch diffusers at 5-foot centres with ridges between, whilst the eastern battery is arranged to operate on the "spiral flow" principle, with two rows of 8-inch diffusers placed at the foot of the wall on one side of the channel. As there was no evidence to demonstrate the superiority of either of these methods, the plant was designed so that one could be replaced by the other, should experience show any definite advantage.

In both batteries, baffle-walls are provided at approximately

100-foot intervals, with transverse diffusers under them to lessen the possibility of direct flow through the channels, and all tanks and channels can be emptied by drain-valves at these points into a 36-inch drain discharging into the Western low-level sewer. The diffuser-stones are of a siliceous material, mounted in cast-iron trays, and can be cleaned by firing.

Two branch air-supply pipes, located in troughs in the tops of the walls, are provided to each unit. The maximum diameter of these branch pipes is 15 inches, and they are reduced as required to 12, 10, 8, and 6 inches. The air-supply to each line is measured by an orifice-type meter, and is controlled by a globe valve. Connexions to the diffusers are 1-inch galvanized mild-steel pipes, with needle-type regulating valves at coping-level. Each valve controls one diffuser, and the air-supply to individual plates can be regulated as required.

Fig. 27.



AERATION-TANKS.

The operating-gallery (Fig. 28, Plate 2 and *Fig. 29*, facing p. 511) houses all the controls of valves and penstocks as well as meters for mixed liquor, compressed air, and return-sludge. The duty of each unit of the aeration-plant is metered and controlled at the outlet end, where the 36-inch venturi-meter and sluice-valve housed in the operating-gallery connect with the mixed-liquor channel. The return-sludge from the final separating-tanks is conveyed by gravitation-mains running in a subway beneath the final separating-tank supply-channels, under the operating-gallery, and discharges through 16-inch venturi-meters into the return-sludge channel, which is aerated, terminating in a hopper at the return-sludge pumping-station.

Air-supply at a maximum rate of 2 cubic feet of free air per gallon of sewage, based on the dry-weather flow, was provided, requiring a total quantity of 60,000 cubic feet per minute, including an allowance for aerated connecting-channels at the rate of 2 cubic feet per square foot of nominal diffuser-area. It is anticipated, however,

that the quantity of air actually used will be considerably less than this figure, probably in the region of 1·4 cubic foot per gallon, based on the dry-weather flow, which, with the tank-dimensions adopted and the diffuser-ratio of one-tenth of the tank area, corresponds closely to a rate of 1·4 cubic foot of free air per minute per square foot of nominal diffuser-area. The normal working pressure is expected eventually to be about 7 lbs. per square inch at the blowers, while the range of pressure provided for is from 5½ lbs. to 9 lbs. per square inch. During the early stages of working the pressure has been about 6 lbs. per square inch, the higher pressure being provided in order to make it possible periodically to increase the pressure for the purpose of keeping the diffuser-stones free from deposit.

Compressed air is delivered from the compressor-house through a 72-inch cast-iron main, which bifurcates at the return-sludge pumping-station into two 51-inch mains, each supplying one battery of aeration-tanks, and being reduced appropriately in diameter down to 33 inches at its end.

Return-sludge has been allowed for at rates up to 50 per cent. of the dry-weather flow, and provision has also been made for the return of effluent at the same maximum rate for the purpose of diluting the influent. The economics of effluent-return remain to be borne out by experience, but, at the expense of a certain amount of power for pumping, it becomes possible to control to some extent the strength of the sewage to be treated. It is well known that troubles due to stale and septic sewage, which occur chiefly in dry summer weather, are usually remedied by the effects of a shower which increases the dissolved oxygen content of the sewage. It has therefore been arranged to return up to 100 per cent. of the dry-weather flow, of which 50 per cent. is return-sludge and 50 per cent. return-effluent.

For this reason the return-sludge pumping-station, situated between the two batteries of aeration-tanks, houses the following plant :—

Four 16-inch return-sludge pumps, normal duty 3,500 gallons per minute against a total head of 3 feet.
Two 20-inch effluent-return pumps, normal duty 7,000 gallons per minute against a total head of 8 feet.
Two 4-inch surplus-sludge pumps, normal duty 300 gallons per minute against a total head of 13½ feet.
One 6-inch drainage pump for the purpose of emptying the final separating-tanks.

All the pumps are driven by variable-speed motors, and by the combination of this factor with the arrangement of pumps selected, it is possible to return any volume of activated sludge from 15 per cent. up to 50 per cent. of the dry-weather flow. The effluent-return pumps, each with a maximum capacity of 50 per cent. of the dry-

weather flow, can be used as standby for the return-sludge pumps, as well as for returning effluent for dilution purposes. Both return-sludge and effluent-return pumps are of the mixed-flow type, which is admirably suited for low heads. The form of impeller is expected to obviate risk of damage to the sludge flocs. The output of the pumps is automatically adjusted to the flow of return-sludge into the suction-wells by float-controlled regulators. Intermittent pumping is avoided by the use of direct-current variable-speed motors, and, owing to the arrangement of pumps and suction-pipes, the supply of return-sludge to each battery of aeration-tanks is separate and independent.

The various flow-rates through the plant are electrically transmitted to the instrument-panel in the return-sludge pumping-station from the various metering points. The panel indicates :—

Settled-sewage flow to each battery ; indicator, recorder, and integrator.
Compressed air ,, ,, ,, ,, ,, ,, ,,
Return-sludge ,, ,, ,, ,, ,, ,, ,,
Surplus-sludge flow from each battery ; ,, ,, ,, ,,
Return-effluent flow to each battery ; indicator only.
Mixed-liquor flow through each unit ; indicator, recorder, and integrator.
Compressed air to each branch air-line ; ,, ,, ,, ,,

Although each of the ten sludge-return pipes from the final separating-tanks is provided with a venturi-meter and indicator, this is used only as a means by which the amount of sludge from the individual rows of four final tanks can be adjusted, no records being made of the flows through the meters. All sludge-, mixed-liquor-, and sewage-meters are provided with connexions to a fresh-water supply-main in order that accumulations of sludge may be periodically flushed out.

Final Separating-Tanks.

The physical process of settlement and separation of activated sludge from purified effluent is effected in the final separating-tanks. The major factors considered in design were, firstly, that the activated sludge to be settled out is light and flocculent, and, secondly, that the quicker the activated sludge is removed from the tank the less chance is there of its becoming septic, to the detriment of the process. If these two factors are satisfactorily dealt with, activated sludge will separate out in good condition, and the effluent will be virtually free from suspended matter.

Upward-flow tanks were adopted, the type being a compromise between deep Dortmund or pyramidal tanks and shallow tanks with nearly flat bottoms, mechanically swept. It is a matter of experience that the deep pyramidal tank produces the best effluent,

whilst the best sludge is produced from the shallow swept tank; for these reasons an attempt has been made to combine the advantages of the two types. The most suitable form of tank for quick construction in comparatively large numbers had constantly to be studied, and a circular tank was evolved with a diameter of 60 feet, a side water-depth of 8 feet, and a conical floor with a 30-degree slope to the horizontal (Figs. 30, Plate 3). A central feed is provided by a 21-inch cast-iron pipe, laid under the floor of the tank, and the vertical feed-pipe ends in a bellmouth 6 inches below normal water-level. This feed-pipe supports a galvanized steel diffusing-box, 10 feet in diameter and 12 feet deep, which projects 6 inches above normal water-level. The kinetic energy of the mixed liquor entering the tank is dissipated on emergence from the bellmouth, and the liquid then travels vertically downwards slowly through the annular space between the diffusing-box and the inlet-pipe. The upward rate of flow in the tank is restricted to allow the activated sludge to settle, the design of the tanks being based on a maximum wet-weather upward rate of flow of 7 feet per hour. This rate corresponds with approximately 1,100 gallons per square foot of tank-area per day, being reduced to one-third in dry weather.

The principle of sludge-removal aimed at was not so much the forcing of the sludge towards the centre of the tank by sweeper-blades, which is virtually impossible in the case of such a flocculent material, but the absolute prevention of settlement on the floors by the frequent dislodgement of the sludge in such a manner that, due to the effect of the sloping floor and of the suction-currents towards the apex of the cone, it would rapidly be discharged from the bottom of the tank through the delivery-pipe. Deep blades, as used for coarser sludge, were not suitable, and thin ribs, not unlike those of an inverted umbrella in arrangement, were adopted. The disturbance desired was to be no greater than that required for the purpose, as any violent action would be detrimental to the settling process, and a maximum peripheral speed of 3 feet per minute was adopted. The diameter of the tanks was fixed at 60 feet, being arrived at from consideration of the aggregate area of tanks required, the general question of the type of sweeping mechanism, and the maximum peripheral speed of the blades. For the latter considerations this size was the maximum desirable. An installation of forty tanks divided into two batteries of twenty was adopted, each battery being conveniently located in relation to a corresponding battery of aeration-tanks.

The design of the sludge-removal mechanism was influenced by the fact that the machines were to be fitted to a large number of moderate-sized tanks grouped closely together, so that steel bridges

spanning them would be unsightly. Eventually a design was evolved which complied with all the conditions. A revolving ring of steelwork of approximately the same diameter as the effluent-weir is supported thereon by rollers fixed to concrete blocks, and is kept in position by vertical guide-rollers fixed over the effluent-channel. The ring is composed of rolled-steel channels braced along the periphery to maintain a true circular shape, and carries a cast-iron rack on its underside. A pinion engaging with the rack rotates the whole ring at a maximum peripheral speed for the scrapers of 3 feet per minute. Eight vertical tubular members are rigidly fixed at equal intervals to the revolving ring, and at the lower end of each of these tubes is hinged a 6-inch by 2½-inch oak scraper, in two straight lengths loosely jointed together. The lower ends of the oak scrapers are all fastened together to keep them in position, and they are also stayed by means of rustless-steel rods supported from the revolving ring by wire ropes. Supplementary scrapers, 2 feet in length, faced with rubber, are hinged from one of the eight main scrapers to give a final cleaning to any small irregularities in the floor. An oak blade fixed to one of the vertical tubular supports is provided to sweep the vertical sides of the tanks, the pressure against the wall being adjusted by weights.

The revolving steel rings are driven in pairs through reduction-gearing by a 1·7-HP. electric motor, chain-type couplings being provided on both driving shafts, so that one ring can be disconnected and stopped, leaving the other in operation. Tests showed that only 0·8 HP. was required to operate one pair of rings.

The tanks are fed from six mixed-liquor supply-channels, each 4 feet wide by 12 feet deep and fitted with air-duffusers. Cast-iron gratings over them act as gangways and are removable to give access to the air-valves. The control-penstock to each tank is 30 inches in diameter, the inlet being bellmouthed from 21 inches to 30 inches to reduce head-loss.

Sludge is withdrawn from the tanks through a 9-inch sludge-pipe (Figs. 30, Plate 3), flowing into an inspection-chamber over an 8-inch diameter telescopic screw-down bellmouth weir, which can discharge up to 50 per cent. of the volume of the influent to the tank. From the chamber it flows to the return-sludge pumping-station through 12-inch and 16-inch return-sludge pipes in a pipe-gallery leading to the lower floor of the operating-gallery. The tanks are emptied through the sludge-pipes into a 12-inch drain connected to pumps in the return-sludge pumping-station. Top water can be emptied by gravity, but the lower portion must be emptied by pump. The purified effluent from the tanks is discharged into branch culverts situated between rows of tanks, and thence through the main effluent-

culvert, passing along the south and east side of the battery of storm-water tanks to the effluent down-shafts at the north-east corner of the works.

Several methods of construction were tried for the conical floors of the final separating-tanks, but circumferential construction-joints were not entirely satisfactory because any irregularity would have caused the radial blades of the sludge-scrapers to miss a portion of the floor. A later method of placing the concrete floors in two thicknesses, one of 7 inches and one of 2 inches, was adopted the floor being laid with radial construction-joints. A stiff concrete mix was used, and it was found that, although the abolition of shuttering avoided many constructional difficulties, the dryness of the concrete during placing made watertightness difficult to attain.

Storm-Water Tanks.

It is common practice in Great Britain to fix the capacity of storm-water tanks as 6 hours' detention of the dry-weather flow of sewage, but there has been a growing tendency in recent years, especially where combined systems of sewerage are in operation, to increase this allowance. In the case of West Middlesex the local systems are separate or partially separate, and because of this fact and of the very considerable balancing effect of the main sewers serving such a large area as 160 square miles, wide fluctuations in peak flows would tend to be eliminated. On the other hand, although the dry-weather flow expected in the early years is only one-half of the ultimate dry-weather flow, it is possible in wet weather to receive a relatively larger proportion of the ultimate flow, on account of deliberate flooding relief. The storm-water tank capacity immediately to be provided was determined as 6 hours of the ultimate dry-weather flow, equivalent to 20,000,000 gallons. The general design of the tanks was dictated by local circumstances. Limits of length of the battery of tanks were imposed by the site; limits of depth were governed by the high-level sewer on the one hand and by the low-water level of the river on the other; and tank-widths by the span of the sludge-removal machines.

Machine-cleaning of the tanks was rendered necessary by the same considerations as those discussed for the sedimentation tanks (p. 513). So important were these considerations that, although such methods had never been previously applied to storm-water tanks, it was determined that they constituted the only practicable solution of the problem. At that time, however, almost all machines operating in Great Britain were of the rotating type, which was too costly to contemplate for storm-water tanks. Six or eight tanks would be required for flexibility of control, and steps were taken to

evolve a new design of machine which would be suitable for the proposed conditions. At this time there came on to the market a machine known as the Mieder, similar to machines successfully operating in Germany, notably at Leipzig and Nuremburg, on a smaller scale. The chief features of the machine are the travelling bridge, spanning a tank rectangular in plan, from which a deep scraper-blade is suspended. Sludge is scraped along the floor of the tank under water into hoppers formed at one end. One of the chief advantages of the machine is that, by means of a transporter-carriage, it is possible to transfer the scraper from one tank to any of the others, thereby making it essentially suitable for the intermittent duty required in connexion with storm-water tanks. By this means a comparatively cheap installation is possible. In the final lay-out of the tanks a scraper span of 75 feet was adopted, and the ultimate design consisted of two batteries each of four tanks 150 feet wide by 230 feet long, with a roadway between the batteries. A longitudinal track-beam, supported on columns, divides each tank into two bays, each measuring approximately 230 feet by 75 feet and having an average water-depth of 12 feet (*Fig. 31*, facing p. 511). Each tank bay is provided with three sludge-hoppers at its inlet end, each measuring 25 feet by 18 feet 6 inches in plan, with minimum slopes of 45 degrees.

Two storm-water feed culverts originate at the storm-water separating-weir and run to the north-west corner of the storm-water tanks. At this point they become open channels, the inner channel feeding the first four tanks, while the outer channel, running the full length of the two batteries, serves the second series of tanks. Connexion from the supply-channels to the tanks is effected by six 42-inch by 30-inch electrically-operated penstocks to each tank, while the feed-channels themselves can be isolated by 8-foot by 7-foot penstocks also electrically operated.

The transporter track lies between these channels and the tanks, and the space underneath it is made to act as a flow-diffuser, ensuring a uniform velocity of entry into the tanks, and avoiding any tendency for eddies to form or for " short-circuiting " to take place. Uniformity of flow across the width of a tank being essential for the best results, these diffusion-galleries are designed so that the flow through them is directed longitudinally into the tanks, no matter at what level the storm-water in the tank may be. The three inlets to each tank bay are only 5 feet 3 inches above tank floor level, and the floor of the diffusion-gallery slopes and so eliminates sudden drops or waterfalls, avoiding splashing and consequent smell as well as preventing sludge from being deposited. The inlet-end wall of the tanks is formed by vertical columns 1 foot 6 inches by 1 foot in

section at 2-foot 6-inch centres, their function being to break up any directional flow of the influent and distribute it both in horizontal and vertical planes in a direction parallel with the length of the tank.

The tank floor falls 12 inches in a length of 190 feet, this being regarded as sufficient for the sludging operation. Although top water-level is 20·0 O.D., the coping level is 24·0 O.D., this freeboard being allowed so as to give an ample margin of safety from floods. The maximum high tide recorded at Isleworth ferry during the past 200 years is 19·27 O.D. Even if such a flood were coincident with the maximum discharge from the works, causing the worst conditions of head-loss in the works and through the twin effluent-conduits to the river, the situation would still be under control, although in such a remote event it would be necessary to isolate the purification-plant for a short time, using the storm-water tanks only until the tide ebbed.

Owing to the compact lay-out of the whole battery of tanks, it was necessary to provide pathways on the main division-walls in order to give safe access along both sides of each tank while the sludging-machine was working. These pathways were made 4 feet 6 inches wide, and, allowing for parapet-walls, the total width of the division-walls became 7 feet 6 inches. Advantage was taken of this width to build two decanting culverts in each wall, of 3 feet 6 inches and 4 feet 6 inches in diameter respectively, which were connected to the tanks by 18-inch diameter inlets spaced at 16-foot 6-inch centres. The roadway division-wall is shown in *Fig. 32* (p. 524).

Decanting can be carried out in two stages; firstly, the top 7-foot layer of storm-water into the top culvert, and secondly a layer 3 feet 6 inches in depth into the lower culvert; the bottom 18 inches is left to be emptied through the sludge-hoppers. Sludge is discharged from the hoppers under hydrostatic head, either by visible discharge into small inspection-hoppers, or directly into the sludge-main. Arrangements have also been made for the pumps to be able to pump back along the suction-main into the hoppers for the purpose of clearing any stoppage that may occur.

It was intended that the tanks should not stand empty in periods of dry weather when there is no storm-water flow, but that, during such conditions, effluent from the final separating-tanks should be diverted into the storm-water tanks. The main advantages of this would be :—

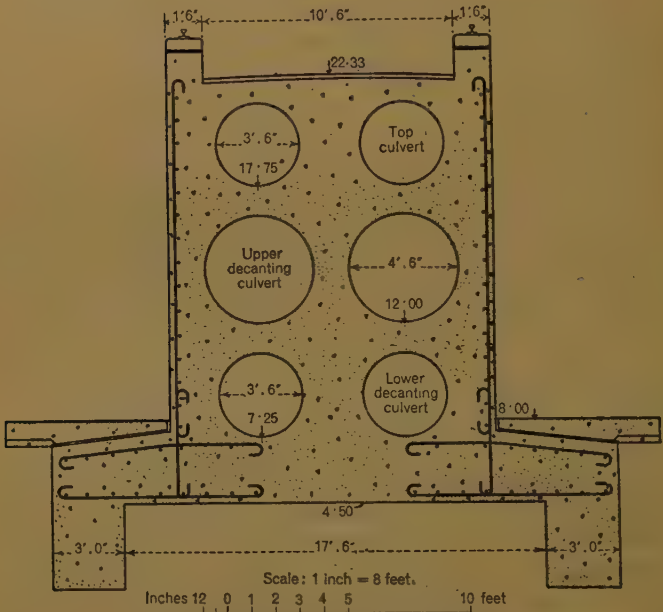
(*a*) Additional detention and mixing of the effluent from the final separating-tanks will be provided.

(*b*) Concrete flooded with water is not subject to the same risk of trouble due to expansion and contraction as when exposed to the weather.

(c) In times of wet weather, the storm-water would enter a full tank in a quiescent manner and proper sedimentation would commence immediately.

(d) Before storm-water escapes over the outlet-weirs it would be necessary to force a quantity of effluent out of the tank, and the mixing of storm-water and effluent during this period would render the discharge of storm-water to the river a more gradual process.

Fig. 32.



CULVERTS IN MAIN DIVISION WALL.

To enable the tanks to be worked at the highest possible level during storms, and on occasions to allow final effluent to be passed through them, weirs were provided at 20·00 O.D. for storm-water and at 16·75 O.D. for effluent. The storm-water weir consists of a wall 18 inches thick, at the base of which are six 40-inch by 27-inch electrically-operated penstocks to each tank, admitting final effluent direct to the effluent-weir at the lower level. The difference between the levels of the storm-water weir and the effluent-weir is caused by head-loss at the maximum flow allowed through the purification-works and by the further loss involved in diverting the effluent through the storm-water tanks. Tank-effluent passing over the

storm-water weir, or through the penstocks in its base, passes in a thin film over a weir 6 feet in width faced with white glazed tiles, and falls into the main effluent-culvert, 15 feet wide by an average height of 15 feet, discharging in turn to the effluent-conduits.

Under the outlet-ends of the tanks is located the main decanting culvert, to which are connected the two decanting culverts of each tank (Fig. 33, Plate 3). Control of decanting is maintained by 24-inch penstocks at the junctions. Under the effluent-weir itself is formed a by-pass culvert 10 feet wide by 6 feet high, which may be brought into use during inspection or repair of the main effluent-culvert. The decanting culvert, by-pass culvert and main effluent-culvert, together with the storm-water- and effluent-weir arrangements, are incorporated in a monolithic structure. Electrically-operated inlet and outlet penstocks on the tanks are so arranged that they can be operated individually or in groups of six. Operation is by push-button, but arrangements are made so that remote control from the control room in the main pumping station can be added at a later date at the expense of laying a cable between the two points.

Construction of Storm-Water Tanks.

The main excavation was started at the northern end of the site early in April, 1933, with diesel-driven Ruston-Bucyrus dragline excavators of $\frac{3}{4}$ cubic yard capacity, and the excavated material was hauled to the spoil banks by lorry, a fleet of 5-ton four-wheel-drive lorries with steel tipping-bodies being used for the purpose. The use of lorries instead of railway track was decided upon because of the long narrow shape of the spoil banks, and very satisfactory results were obtained, the running of the lorries assisting in the consolidation of the banks. This work was favoured by a dry summer and dry winter. A temporary road, 15 feet wide, formed of a 7-inch concrete slab made with ballast excavated from the site was laid around three sides of the area and was removed at the end of the operations. Branch roads were formed as required to give access to the tips and to the tanks.

The main excavation-cut was made from ground-level, which ranged from 24 O.D. at the north and 35 O.D. at the south end of the site, down to approximately 10 O.D., and the material was dumped into lorries running on the temporary road at original ground-level. Over most of the area a sandy gravel overlay London clay, but all foundations were made in the clay at the minimum depth anticipated. The gravel varied considerably in its composition but at no time was excessive water encountered, although the

excavations were carried well below tide-level in the river Thames about ¾ mile away.

A portion of the site overlapped the area dredged under contract "M.1," over which had been deposited a layer of soft silt approximately 3 feet deep. The silt was difficult to handle in its original semi-fluid condition, and it was found necessary to tip a layer of solid excavated material on the top of it to give a reasonable foundation on which the excavators could stand. The admixture of excavated material with the silt made handling possible and reduced the tendency to slip when placed in the spoil banks. Excavation below 10 O.D. for floor and wall foundations and for the sludge-hoppers was carried out as a separate operation, the excavators and lorries standing on the lower level. Ramps were made to give access to the temporary concrete road at ground-level, which also provided access for construction-materials. The average formation-level for concrete floors was 7 O.D. and this excavation was done with a ½-cubic-yard skimmer-excavator. A back-trencher excavator with a ½-cubic-yard bucket was used for excavating trenches for wall foundations down to a level of about 1 O.D. Excavation for sludge-hoppers at the west end was done by hand with pneumatic clay-spades, a 2-ton steam rail crane being used to handle the spoil into lorries.

Site limitations played an important part in the arrangements made for spoil-bank tipping. Top soil set aside from "M.1" contract had already been dumped along the line of part of the permanent spoil banks, and "M.2" contract imposed the duty of spreading this soil on the surface of the embankments. This operation involved considerable ingenuity to avoid excessive handling, and influenced the decision to transport spoil to the banks by lorry rather than by rail. Spreading and consolidation of the tipped soil was done by tractors fitted with bulldozer plates used to maintain a level surface fit for the lorries to run over. Consolidation attained by this method was unusually good.

A certain amount of excavation and removal to tip by 2-cubic-yard scraper-excavators or graders drawn by tractors, was done at the southern end of the site, but the spoil banks were not well arranged to suit this method of travelling in circular paths and it was therefore discontinued. Gravel had to be scarified before the graders could deal with it, but looser material was well suited to the type of plant.

All concrete was mixed in a batcher plant driven by electric motors; aggregates were automatically proportioned by weight, tell-tale dials indicating to the operator whether or not each batch was correct. The aggregates were delivered by lorry up a

ramped road formed on the embankment and over a timber gantry into 50-ton overhead bins, and cement was added in standard 1-cwt. bags. An automatic water-gauging tank was fitted, but, owing to variation of the water-content of the aggregates, was not found to be of great value in helping to produce a uniform slump in the mixed concrete. The concrete was transported from the mixer to the tanks by lorry. For foundations and floors the concrete was carried in bulk, but for walls it was carried in special skips taken in lorries, lifted by crane and poured behind wall-shutters through spouts in the skips. In addition to regular cement-testing, 6-inch cubes were frequently made from the concrete mixed on the site; aggregates were tested for grading and cleanliness; and slump tests were made at frequent intervals during concreting operations.

The type of construction provided good opportunities for the use of travelling steel formwork, and some interesting examples were provided by the large division-wall faces and the internal surfaces of the three main culverts which form the outlet end of the tanks. The division-wall forms consisted of steel bogies running on light tracks on each side of the wall; on them five light steel portal-frames were erected, which in turn supported the steel face-shutters, the latter being provided with adjusting screws for setting plumb and to line. The shutters were 5 feet high, three lifts completing the full height of the wall. Spreading of the shutters was prevented at the bottom by strutting the bogies from a line of steel pins cast into the concrete sealing-coat under the floors, and at the top by the rigidity of the head of the portal-frame, which was also used for raising the shutters by chain blocks from lift to lift. A similar type of travelling shutter was constructed in timber and steel for the open supply-channels and for the line of baffle-columns at the inlet end of the tanks. Forms for the culverts consisted of steel frames on flanged wheels, with a flat steel shutter above for the underside of the roof, and hinged side-plates, the tops of which were curved to form the cove between wall and soffit. The form was run into position on rails and jacked up to the correct level, the side-pieces swung out by adjusting screws until plumb, and when in position these three forms provided for the complete internal shuttering of a 26-foot length of culvert. The necessity of mobile shuttering for these culverts is emphasized when it is realized that the outlet end of the storm-water tanks consists of a main effluent-culvert 15 feet by 13 feet 6 inches, a by-pass culvert 10 feet by 6 feet 6 inches, and a decanting culvert 6 feet by 6 feet 6 inches, all three running the full length of the battery of eight storm-water tanks (about 1,270 feet). The placing of concrete in the three culverts was facilitated by a steel travelling gantry spanning the 50-foot width of the three culverts

and supporting a 2-ton petrol-driven rail crane to handle the concrete-skips (*Fig. 34*).

The main constructional plant comprised :—

Two $\frac{3}{4}$-cubic-yard excavators with fitments to convert from dragline to skimmer ;
six $\frac{1}{2}$-cubic-yard excavators, convertible to dragline, trencher or skimmer ;
two scraper-excavators, or graders, drawn by tractor ;
four bulldozers, or mechanical spreaders and levellers ;
eight four-wheel-drive 5-ton lorries ;
three 5-ton lorries ;
twenty 2-ton trucks ;
four 2-ton mobile petrol cranes ;
one „ „ „ „ on gantry for culverts ;
two light mast cranes ;
two 2-ton steam rail cranes ;
one batcher and mixer plant.

Sludge-Disposal.

The Mogden site is limited and is quite insufficient in area for sludge-drying and disposal. A supplemental area for this purpose was necessary for that reason alone, not to mention the opposition which would have been created, largely on sentimental grounds, had it been proposed that sludge should be exposed in the open at Mogden, in the midst of an urban district. Nevertheless, power is required at Mogden, the development of power from sludge gas is economic, and partially-digested sludge is easier to pump than crude sludge or fully-digested sludge. These were all strong arguments in favour of the decision to separate the two stages of sludge-treatment plant, by constructing heated primary digestion-tanks with gas-recovery installation at Mogden, and carrying out secondary digestion, sludge-drying, and final disposal by tipping at Perry Oaks. In any case, however, prudence would have dictated that sludge storage-capacity at Mogden was essential. Pumps, motors, power and pumping mains are not infallible, and the necessity to be able to impound sludge at Mogden before pumping would have followed in any case the decision to pump sludge 7 miles to Perry Oaks.

The crude sludge produced from the sedimentation-tanks, both primary and secondary, and from the storm-water tanks is conveyed to the raw-sludge pumping-station normally by gravity. Situated at the east end of the primary tanks, this pumping-station is located in a central position so that in no case is an excessive length of suction-main involved. On the delivery side of the station, a 12-inch cast-iron main runs southwards for the length of the storm-water tanks and thence westwards at the rear of the compressor-house to the sludge-digestion tanks. From this point the sludge-pumping main

Fig. 34.

STORM-WATER TANK NO. 1, NORTH-EAST CORNER, UNDER CONSTRUCTION.

Fig. 36.

PRIMARY SLUDGE-DIGESTION TANKS.

Fig. 39.

POWER- AND COMPRESSOR-HOUSE.

Fig. 40.

MAIN PUMPING-STATION.

to Perry Oaks is laid at the side of the river as far as the north end of the works, and this made practicable the addition of a main along the north side to form a closed circuit or ring-main. By connexions to this ring-main at various points, considerable flexibility of control and operation is effected; it is possible to avoid interruption to pumping by reason of a burst or the making of a new connexion, and the chance of blockage is reduced. All the sludge-pipes, whether suction or pumping mains, are in this way brought within the influence of the main sludge-pumps, and hence can be subjected to a pressure of 150 lbs. per square inch, should that be necessary.

For the reception of crude sludge delivered from the raw-sludge pumping-station, sludge-thickening tanks are provided where settlement and stirring may be used to reduce the water-content before the thickened product is pumped into the sludge-digestion tanks. After primary treatment in the latter tanks for a period approaching 1 month, the partially-digested sludge is drawn off and pumped to Perry Oaks for completion of the digestion-process, drying and disposal by tipping.

The total capacity of the primary digestion-plant provided is 10,375,000 gallons, allowing 1·66 cubic foot per head of the 1,000,000 population initially served and about 1·30 cubic foot per head when the population reaches 1,250,000. Preliminary design and estimate indicated that circular tanks were considerably cheaper than any other type, and that a diameter of approximately 70 feet was the economic size for the given conditions.

Constructed partly below natural ground-level and partly above it in embankment, the twelve digestion-tanks (Fig. 35, Plate 3) are located at the south end of the site, just west of the river. The filling around the tanks is 13 feet high above the original ground-level, but the embankment rises 15 feet above them, being about 28 feet in total height (*Fig. 36*, facing p. 528). In this way, the tanks are protected from cold winds. The thickening-tanks and main sludge-pumping station are located to the north of them, just outside the toe of the embankment. They are each 35 feet in diameter and 20 feet in side-water depth, with central inlets near the full water-level and peripheral weirs and channels for supernatant water. The mechanism consists of a central driving drum, surrounding the central column, and a driving motor mounted on the column. Blades on four arms attached to the drum move thickened sludge to the central sludge-outlet in the bottom, and galvanized mild-steel pipes mounted vertically on the arms at 9-inch centres are rotated through the sludge.

Thickened sludge is drawn direct from the annular sumps in the tank-bottoms by the centrifugal pump in the main sludge-pumping

station and forced to the digestion-tanks. Supernatant water gravitates to a sump under the pumping-station and thence to the low-level sewer under the works, but scum formed on the tanks is swept by the mechanism to chambers in the side-walls and is pumped with thickened sludge to the digestion-tanks. Sampling can be carried out as in the digestion-tanks.

Both thickening tanks are housed in a building 88 feet long by 47 feet 6 inches wide, with a hollow-tile flat roof. Air for ventilation is drawn in at floor-level at the west end and exhausted at roof-level at the east end by a 15-inch electric fan.

Of the twelve digestion-tanks, eight have hopper bottoms and spirally-guided gasholder tops, whilst the other four tanks have nearly flat bottoms, fixed roofs and stirring mechanism.

Each gasholder-tank is of 146,000 cubic feet capacity, having a nominal diameter of 70 feet and a side-water depth of 32 feet, with a 2-foot freeboard. The conical floors have a slope of 30 degrees, giving a central depth of about 54 feet. The gasholders are of the spiral-carriage type, eight pairs of rollers being fixed to the tank wall for the purpose of guiding the spiral rails mounted on the gasholder walls. The material chosen for the construction is copper-bearing mild steel. In its lowest position each gasholder rests on a ledge inside the tank, 12 feet from the top, and can rise 9 feet 6 inches to hold 37,000 cubic feet of gas. Having a weight of about 50 tons, the holder stores its gas at a pressure of 5½ inches of water. Two Warren trusses of copper-bearing steel are built into each tank below the lowest level of the gasholder at 15 foot centres to carry all pipework entering the tank.

The four stirred tanks have fixed roofs and floors which are nearly flat. They are 70 feet in diameter, with a side-water depth of 32 feet, each having a capacity of 123,000 cubic feet. The mechanism used for stirring the sludge, which is driven by motors mounted on the fixed roof, consists of a central driving drum revolving round a fixed column and carrying four arms near the bottom and two at the top. On the lower four arms are ploughs to give the sludge a slow motion towards the centre of the floors, and from the upper two arms thick chains are suspended to act with the arms themselves as scum-breakers and sludge-stirrers. Flame-proof motors, each of 2½ HP., drive the central drum. Gas is collected in small gas-domes 2 feet 6 inches square, built into the roofs, but providing no storage for surplus, which can, however, pass into the gasholders of the other eight tanks. Each roof has an access-manhole, which, like the driving drum, gas-dome and all outlets, is provided with a water seal.

The full battery of twelve tanks is divided into three sets of four, and the observation and control of the operation of each set is

effected from control-chambers located in the space between them. Instead of refilling excavated material completely around all of the twelve tanks, use was made of the space between the two lines of six tanks to construct a pipe-gallery which runs under the three control-rooms referred to. These chambers are bounded by the contiguous circumferential walls of four tanks connected by galleries running from the main sludge pumping station to chamber No. 1, from there to No. 2 and again on to No. 3, the whole being superimposed on a pipe-gallery constructed in a basement 9 feet 9 inches below.

The heating of the tanks is carried out by coils of 2½-inch galvanized mild-steel pipes, generally placed at 18-inch centres. In the stirred tanks the pipes are in three banks, one of six and two of five coils, supported by brackets fixed to the walls of the tanks; in the gasholder tanks there is one bank of six pipes bracketed from the wall and a coil of ten pipes in the form of an octagon suspended from the truss.

Sludge is delivered to the tanks through two 12-inch cast-iron mains running from the main sludge-pumping station through the galleries and chambers the full length of the tanks. A connexion is made to each tank from each main, and at these connexions breeches-pieces interconnect the two 12-inch mains where the pipe enters the tank. In the gasholder tanks the inlet-pipe is carried on the truss and divides to discharge vertically upwards through bell-mouths at four points about 3 feet 6 inches below the full water-level, traps being formed in them to prevent the escape of gas when the sludge-level is below the bell-mouths. In the stirred tanks, the inlet-pipe is carried up the wall of the tank on to the roof and is divided to deliver to the tank at three points. The discharge is downwards through the roof, the outlet being fitted with a cone-shaped tray to improve distribution.

The outlet for the sludge in all tanks is from the centre of the floor. In the gasholder tanks the outlet-pipes are supported on the floors of the tanks, extending from the bottoms of the cones up to their point of exit through the wall into the control-chamber. The sludge in the stirred tanks is moved by means of the blades of the machine to a small sump built around the centre column, and drawn off through a pipe running under the floor. All the outlet-branches feed into a common 12-inch main, running the full length of the galleries and chambers to the reciprocating pumps in the main sludge-pumping station.

Provision has been made to re-circulate the sludge in the tanks and to seed the incoming sludge with partly-digested sludge, a 500-gallon-per-minute centrifugal pump located in Chamber No. 2 being used for the purpose. The suction side of this pump draws

sludge either from the bottom of the four stirred tanks or, in the case of the gasholder tanks, from the level of the top of the cone at about the centre of the tank through a special pipe provided for the purpose. The delivery of the pump is direct to one of the raw-sludge delivery-pipes. Thus partly-digested sludge can be circulated from tank to tank, circulated within the same tank, or intimately mixed with the incoming raw sludge before the latter enters a tank.

Separated water is drawn off from the gasholder-tanks at five different levels, the top one at 2 feet below the surface and the others at intervals downwards of 3 feet. In the stirred tanks the main supernatant-draw-off is by an overflow 2 feet 6 inches from the underside of the roof. Water discharges into the chamber, and thence through a pipe, the top of which is set at full water-level, the stirring mechanism being relied upon to bring the separated water to the top. Two additional draw-off points are also provided at lower levels. Supernatant-water-pipes are sealed against gas-leakage, in the event of the sludge-level being below the mouth of the pipes. In the case of the gasholder tanks, this is achieved by means of vertical bends in the pipes to form traps, while in the stirred tanks similar traps are formed by special mild-steel seal-boxes attached to the wall covering the openings.

All pipes passing through the walls of the tanks are lead-caulked on both sides of the wall into the double-ended sockets of special cast-iron puddle-flanged sleeves, built into the wall during construction.

Waste heat from the engines in the compressor house is used in a closed hot-water circuit which passes through the coils in the sludge-tanks to keep the temperature of the sludge at about 80° F. The direction of flow in the several heating coils within the tanks can be controlled at will from the interconnecting "grids" at each tank in the control-chambers, or the coils can be used in series or in parallel. All hot-water heating pipes outside the tanks are insulated with 1½ inches of cork, sealed with emulsified bitumen.

Provision is made for sampling the contents of the tanks at various levels, as well as the incoming sludge and the supernatant overflow-water.

The main sludge-pumping station is about 50 feet long and 26 feet 6 inches wide, the general floor-level being about 17 feet below natural ground-surface. In it are installed two reciprocating pumps, each of the two-cylinder double-acting type, to pump the partially-digested sludge to Perry Oaks against a maximum head of 150 lbs. per square inch. They are driven by 58-HP. variable-speed d.c. motors, to give maximum discharges of 665 and 500 gallons per minute respectively. The equipment includes also two centrifugal pumps, one for the pumping of thickened sludge from the thickening-

tanks to the digestion-tanks, and the other for returning supernatant water from any of the digestion-tanks to another digestion-tank for further settlement, with the object of obtaining a supernatant liquor that will give the least possible trouble to the aeration-plant. These pumps are of the open-impeller type, and have a mean discharge of approximately 500 gallons per minute.

The pipework in the control-chambers and pumping-station is arranged so that any sludge or supernatant-water pipes can be connected to either the suction or delivery side of the reciprocating pumps to obtain further flexibility, and all pipes can be readily cleared of blockage.

So many service pipes and cables from the compressor-house to the main sludge-pumping station had to be laid beneath the river, or, in the case of sludge-mains, for part of that distance, that a subway was constructed to facilitate both laying and inspection. In addition, use is made of it as a footway tunnel between the two buildings, communication between which was otherwise not easy. The subway is 7 feet 6 inches high and 6 feet wide, built approximately on a level with the lower floor of the main sludge-pumping station, with which it communicates directly. In it are housed the main sludge-delivery pipes to Perry Oaks, the raw-sludge delivery from the raw-sludge pumping-station, the hot-water inlet and outlet pipes, the two gas-pipes, a small pipe for town's water supply, electric and telephone cables, etc. With the exception of the water pipe, all in this subway are 12-inch cast-iron pipes, fitted with "Victaulic" joints for easy removal if necessary.

In the east end of the pumping-station is a gas-control chamber 14 feet by 6 feet, isolated from the adjacent structure and housing control-valves, water-traps, flame-traps, and branch valves for waste gas. A tapping is also made to each pipe and connected to a CO_2-recorder mounted in the main building on the loading-platform.

Considerable thought was given to the inclusion of ample measuring and metering devices, both for the more efficient operation of the plant and also for the collection of data affecting the design of any further extensions. Sludge-meters are installed on the raw-sludge delivery, the thickened-sludge delivery, the recirculated-sludge pipe, and the supernatant-overflow. The outgoing sludge pumped to Perry Oaks can be measured positively in the tanks. Gas is metered on the 4-inch branch to each tank. As it is not unlikely that in the stirred tanks there will be a backflow of gas each time sludge is withdrawn, meters reading the flow in both directions are installed. The total production of gas is measured at the compressor-house, and hot water circulated in each individual tank is metered by inferential meter at the outlet side of the circuit. The tempera-

tures of the inlet and outlet hot water in each coil are also measured by ordinary mercury-in-glass thermometers set in thermometer-pockets tapped into the hot-water pipes immediately before they enter the tank. One mercury-in-steel distance recording-thermometer is installed in each tank for the purpose of indicating sludge-temperatures. In the gasholder tanks these are placed at the centre of the tank about 3 feet from the top, but in the stirred tanks, owing to the interference of mechanism, they are bracketed about 5 feet from the wall.

The levels of the gasholders are indicated in the chamber by pointers, moved by wires attached to the holders, and the level of sludge in the gasholder-tanks is indicated by a reaction type of pressure-gauge using compressed air. In addition to measuring the quantity of gas leaving the plant, the quality as indicated by the CO_2-content is measured and recorded.

Special precautions have been taken to prevent trouble caused by escape of gas; as mentioned elsewhere, all openings to tanks and pipes through which gas might escape are either sealed or trapped; all motors and electric fittings, either on the top of the stirred tanks, or in the control-chambers and galleries, are flame-proof; flame-traps are provided on both 12-inch gas lines in the gas-control room; attention has been paid to the ventilation of the control-chambers and galleries; and in each control-chamber has been placed a 10-foot-square roof-light with adjustable fanlights, whilst one end of the gallery is open to the air. Smoking and the use of naked lights in proximity to the digestion-plant is strictly prohibited.

Power.

The ultimate maximum power-requirement of the whole works when serving a population of 2,000,000 persons was estimated at 6,916 B.HP., whereas, at the inception of the scheme, it was assumed to be only 4,400 B.HP. Some of the load is continuous, but the aggregate is liable to considerable variation. Summarized, the total requirements are made up as follows:—

	Present B.HP.	Future B.HP.
Main sewage-pumping station, dry-weather-flow pumps, etc.	480	960
Return-sludge pumping-station	96	192
Raw-sludge pumping-station	48	96
Main-sludge pumping-station	48	96
Auxiliaries	34	72
Compressor-house and works lighting	94	100
Compressors	1,800	3,600
Storm-water pumping	1,800	1,800
	4,400	6,916

Storm-water pumping, governed only by that uncertain factor, the weather, is the major cause of the peak loads, the incidence of the other loads being more readily anticipated, and even controlled in the case of power for the many accessory installations. For the initial period, a load of about 2,700 B.HP. was assessed as the probable demand over lengthy periods, the total load fluctuating chiefly as the storm-water pumps were required or not. The problem to be solved, then, in the design stages was to select the type of power to give economic results throughout all stages of expansion of the works, and thereafter.

Full utilization of the natural power available on works of this character must obviously be studied from the economic point of view before the purchase of power from outside sources can be considered. From the digestion of the sludge produced by works serving 1,000,000 persons, at least 500,000 cubic feet of sludge-gas per day may be expected. When such sludge is predominantly domestic in origin, an efficient plant may be relied upon to produce more than this, and 625,000 cubic feet of gas per day was not an unreasonable estimate of the volume which might be expected. The calorific value of this gas is known to be about 650 B.Th.U. per cubic foot, so that the available energy is at least 16·5 million B.Th.U. per hour, or 1,750 B.HP.

To apply this power of 1,750 B.HP. towards the total normal load of 2,600 B.HP., and to find an economic form of power for the balance of 850 B.HP. plus the additional peak loads, was the natural state into which the problem then grew. The methane gas derived from the sludge could be used in three ways, namely (i) by burning it under boilers and heating water to keep the sludge at the requisite temperature, (ii) by using it to drive internal-combustion engines and applying only waste heat to the sludge-tanks, or (iii) by burning the gas under boilers to produce steam power and applying the exhaust-steam to the heating of sludge. Provided the waste heat from methods (ii) or (iii) is sufficient for heating the sludge, it would be extravagant to use gas itself for this purpose, as in method (i). Having regard to the intermittent requirement for storm-water pumping, which may represent a heavy demand on only a very few occasions in the year, it is obviously uneconomic to provide the necessary gas-storage capacity to make use of this fuel, even if any surplus were available for the purpose.

These considerations resulted eventually in a plan to locate the power-house at the south end of the site near the sludge-digestion plant, to drive air-compressors and electric generators by gas-engines, to utilize all waste heat from the engines to heat the sludge and hence accelerate the rate of digestion, to transmit electric power from the

generators to the many points on the works where power is required (including the dry-weather-flow sewage-pumping), and to make up only any possible deficiency in gas-power by oil-power, whence again waste heat would be available for sludge-heating.

Apart from other aspects of the choice of power for driving storm-water pumps, such as the necessity for immediate availability, comparative estimates showed that oil-power was most economic in all the circumstances. For instance, the annual saving estimated by using oil for storm-water pumping instead of electricity purchased from outside sources was £3,600, taking into account the cost of all plant and of capital works, such as differences in sizes of buildings. Compared with steam-plant an even greater saving could be shown.

For the generation of the main power-load gas would not be available from the sludge for some time after the works were due to come into operation, and it was therefore decided to instal diesel-oil engines, convertible to running on gas as and when it should become available. By this means, the uncertain factor of the volume of gas to be actually obtained, and of the fluctuations in that volume, would be entirely overcome. Engines of the requisite size for the dual duty had not then been manufactured, although the principles were well enough established. No new problem presented itself in driving electric generators by internal-combustion engines of this size, but the problem of driving turbo-blowers running at much higher speeds than the engines was different. Here practically no experience was available, and it was, therefore, necessary to give very careful study to the problem involved. Owing to the great difference in speed between a normal internal-combustion engine and a turbo-blower (the ratio of these speeds being of the order of 10 : 1), it was necessary to introduce increasing-speed gears into the drive. To ensure smooth running, however, it was necessary to protect them from the fluctuating torque of the engine, and particularly from any reversals of torque. This was accomplished by introducing an hydraulic coupling between the gears and the engine fly-wheel so as to smooth out such torque-fluctuations. The coupling also has the advantage of facilitating the starting of the engine, because it does not transmit any appreciable torque until the engine has reached firing speed.

Assuming the whole of the plant to run on oil, the annual cost, taking all capital, maintenance and running charges into account as well as interest on capital, would be £33,500. The comparative cost of performing the same work with electricity bought from an outside authority would be £41,500, so that the production of power on the site from oil fuel represents an annual saving of £8,000.

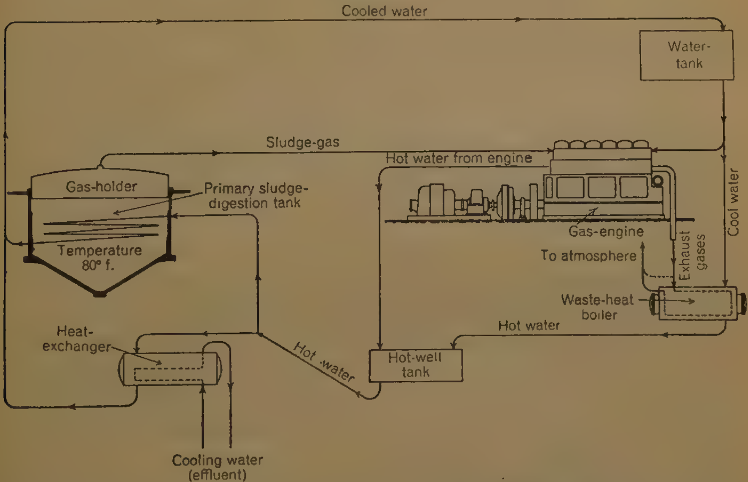
Should enough gas be available for the maximum duty estimated, this annual saving rises to at least £20,000.

The desirability of variable-speed motors for different duties on the site led to the adoption of a three-wire direct-current system, making available supplies at either 460 or 230 volts.

Waste-Heat Utilization.

Energy normally wasted by internal-combustion engines as heat can be put to useful purpose where sludge-digestion plant is in operation. Owing to the comparatively low natural temperatures obtained in practice, the sludge requires heat from an external source in order to maintain the digestion-process at the optimum tempera-

Fig. 37.



ture of about 80° F. Of the total available thermal energy supplied to the engines in the form of gas, approximately two-thirds is rejected in cooling water or exhaust gases, and part of this is then available for heating the sludge. In this manner a closed circuit is developed by which raw sludge, in itself a waste product, is sufficiently raised in temperature by engine waste-heat to produce the power-gas requisite for running the engines.

For the purpose of this heat-recovery (*Fig. 37*), the exhaust-gases are passed through waste-heat boilers to produce hot water, which is discharged to the hot-well tanks, whilst the cooling water from the engine-jackets is delivered direct to these tanks, which are located in the basement of the south annexe. The hot water is circulated by pump through the closed heating-coil circuits of the digestion-

tanks, and is returned to the compressor-house where it is stored in the receiving tank on the gallery of the south annexe. From here some of the flow is again available for cooling of water-jacketing systems, whilst the remainder is diverted through the waste-heat boilers. Losses of water in this circuit are made up from the reserve tank of Metropolitan Water Board's water in the north annexe.

A secondary circuit is provided for cooling the water of the primary circuit during warm weather when the sludge may not require all the available heat. Instead of being passed through the coils in the sludge-tanks, the cooling water may then be diverted through heat-exchangers in the southern annexe, where a supply of purified effluent is available as cooling water. Alternatively, water from the Duke's river may be similarly used for this purpose.

Power- and Compressor-House.

The whole of the power- and compressor-machinery, together with waste-heat and auxiliary equipment, is housed in the power- and compressor-house at the southern end of the site.

The main floor is 330 feet long by 72 feet wide, while the south annexe is 300 feet long by 26 feet wide. There is also a small north annexe for offices, etc.

On the main floor (Fig. 38, Plate 3, and *Fig. 39*, facing p. 529) are located the main engines, compressors and generators, spanned by a 12-ton electric travelling crane. Below ground-level, running longitudinally along the north and south sides, are 14-foot by 12-foot service trenches carrying electric cables and service pipes for gas, oil, air, and water. At a still lower level is a 9-foot 6-inch by 9-foot "ring" air-suction duct, to which are connected the suction-pipes of the turbo-blowers, fitted with silencers. At ground-level, south of the south annexe, are air-intake down-shafts behind vertical air-filters. The down-shafts communicate directly with the "ring" air-suction duct below the main floor.

In the south annexe is housed the waste-heat utilization equipment, above which on a gallery are daily fuel-oil service tanks and a reserve-tank for cooling water for the cylinder-jackets. In a basement 13 feet beneath the annexe, the hot-well tanks, hot-water circulating-pumps, fuel-oil service pumps, and a store for lubricating oil are accommodated. Purified effluent for cooling is provided in an effluent-sump at the east end of this basement, the sump being connected by culvert to the main effluent-culvert just south of the final separating-tanks. This water is not used in the primary engine cylinder-jacketing system, but may be used in summer for cooling purposes should the sludge-tanks not require all the waste heat.

Compressed air from the turbo-blowers is conveyed in cast-iron pipes through the northern pipe-duct to a basement under the north annexe, where it is metered before being delivered to the 72-inch compressed-air main leading to the aeration-tanks. In the same basement is the office-heating installation. Three fuel-oil storage tanks of a total capacity of 100 tons are located below ground south of the building, and are connected to the basement under the south annexe by a pipe-duct.

Although about forty boreholes had been sunk to the level of the London clay all over the site, it was advisable to have even more information about the ground in which the foundations of this building were to be constructed, and therefore eight more bores were put down in the area to be occupied. Even so the information in this particular case was misleading, and when the site was excavated it was found that the ground was unsuitable and that all stanchion-bases and heavy foundations would have to be taken down to clay level. This necessitated complete redesign of the substructure, engine-bases, and ducts during the carrying-out of the contract.

All underground construction was of concrete, stanchion-bases and engine-foundations being mass-concrete, and ducts and walls reinforced. Above ground the building is a steel-framed brickwork structure with flat concrete roofs, the main span being carried on steel trusses.

Mechanical and Electrical Plant in Power- and Compressor-House.

Location.	Plant.	Brief description.
Main engine-room.	Six blower sets.	Each comprising 6-cylinder 650-HP. (550-HP.) diesel (gas) engine driving turbo-blower through hydraulic coupling and increasing gears. Blower capacity 15,000 cubic feet of free air per minute, working pressure 7 lbs. per square inch.
	Four generator sets.	Each comprising 6-cylinder 650-HP. (550-HP.) diesel (gas) engine direct-coupled to a d.c. generator of 450 kilowatts output.
	One emergency lighting-set.	3-cylinder 250-HP. diesel engine direct-coupled to d.c. generator of 175 kilowatts output.
	Two balancer sets.	For 3-wire d.c. system.
	Two auxiliary air-compressor sets.	One driven by 10-HP. d.c. motor and one by 10-HP. diesel engine.
	Main switchboard.	

Mechanical and Electrical Plant in Power- and Compressor-House—continued.

Location.	Plant.	Brief description.
Ground floor, south annexe.	Six heat-exchangers.	To cool engine circulating-water in warm weather. Cooling medium is purified effluent.
	Eleven waste-heat boilers.	Water from receiving tanks heated by engine exhausts and discharged to hot-well tanks.
Basement, south annexe.	Two cooling-water pumps.	One driven by 50-HP. diesel engine and one by 50-HP. d.c. motor. Pump-duty 1,500 gallons per minute.
	Two circulating-pumps.	One driven by 50-HP. diesel engine and one by 50-HP. d.c. motor. Pump-duty 1,500 gallons per minute.
	Oil purifiers.	2-inch centrifuges, pumps, tanks, etc., for oil fuel.
	Two fuel-oil transfer pumps.	One driven by 6-HP. diesel engine and one by 6-HP. d.c. motor.
Basement, north annexe.	Two sets of heating boilers.	Oil-fired.
Gallery, south annexe.	Receiving tanks.	In circulating machinery.
	Ten fuel-oil tanks.	Daily service tanks of 24 hours' capacity.

Pumping-Station.

The low-level area of the drainage district consists generally of Chiswick, Brentford, parts of Isleworth, Twickenham, Teddington, Feltham, Sunbury and Staines, and is served by two main low-level sewers, the Eastern and the Western. The sewage discharged from this area is approximately 25 per cent. of the total flow received at the works, and after being conveyed by gravity is lifted at the main sewage-pumping station.

The flow in dry weather was expected to vary from about 10,000,000 gallons per day at the inception of the works to about 17,000,000 gallons per day ultimately, and the wet-weather flow when the sewers discharge their maximum volume is about 139,000,000 gallons per day. The pumping plant installed is sufficient to pump this quantity, because conditions approaching the ultimate maximum flow may be experienced irrespective of the population resident in the area drained. As time elapses the apparent reserve will diminish, but no change in plant capacity is expected to be made.

A workshop, stores, mess-room, garage, carpenters' shop, smiths'

shop, and station-superintendent's offices are all housed in the same building, which is L-shaped in plan.

Pumps are installed at a low level in a dry well alongside the wet well, with suctions built into the dividing wall, and are operated by prime movers at or near ground-level (*Fig. 40*, facing p. 529, and Fig. 41, Plate 3). The dry-weather-flow pumps are electrically driven by motors vertically over them and at a height of about 64 feet above them, the storm-water pumps being similarly driven through bevel-gears at ground-floor level by diesel engines. Other equipment in the main pump-room includes automatic starters, switchboard, control-board, effluent-pumps, and passenger lift to the pump-well.

An annexe contains fuel-oil tanks, heat-exchangers, exhaust-pipes and chimney, fuel-oil pumps and motors, temperature-indicators for the diesel engines, hot-well tank, auxiliary compressors, and various auxiliaries.

The floor-level of the station is 34·50 O.D., the lowest invert-level of the wet well is — 37·00 O.D., the inverts of the low-level sewers are — 29·75 and — 30·25 O.D., and the invert of the discharge-culvert is 19·25 O.D.

The suction-sump or wet well is 151 feet long by 10 feet wide and is separated from the pump-well or dry well by a 3-foot 6-inch wall. Discharging into it at the south end is the Western low-level sewer, 7 feet in diameter, and at the north end the Eastern low-level sewer, 5 feet in diameter, each sewer being controlled by a penstock, operated by hand or by means of a portable electric drive. Sewage from both sewers must pass through bar-screens having 4-inch openings before reaching the pump-suctions, and a platform is provided for access to these screens at each end of the suction-sump. A lift, capable of carrying 2,500 lbs. at a speed of from 50 to 70 feet per minute, is provided to raise coarse screenings and any large floating body to the surface. When it becomes necessary to clean the suction-sump, or to empty it for inspection, the penstocks controlling the two low-level sewers are shut and the suction-sump emptied by No. 1 dry-weather-flow pump, which for this purpose is provided with a suction-pipe drawing from a sump below the general level of the floor. Flushes towards this sump can be effected by means of large quantities of water from a 4-foot 3-inch drainage-tunnel connected to the sump from the bottom of the two effluent-shafts at the north-east corner of the site. This tunnel at its outlet is controlled by a 36-inch penstock, electrically operated and itself controlled by push-button either from the suction-sump or from the side of the dry-weather-flow pump-starters. A master key provides that only one control can be worked at a time. This drainage-tunnel also provides effluent for cooling water for the

diesel engines, and for hosing down platforms, screens, etc., in the suction-sump.

The pump-well is 151 feet in length by 15 feet in width, and is occupied by the six dry-weather-flow and six storm-water pumps, float-tubes and gear for the control of the dry-weather-flow pumps, the 36-inch effluent-penstock, effluent-pipes and pumps and other auxiliaries. Ample space has been provided for the pumps and for the main gangway, and lighting and heating is provided. Access to the pump-well is by stairway or by lift. Gangways are constructed in the pump-well at 10-foot vertical intervals to give access to vertical-spindle bearings, venturi-meters, etc.

All rates of flow up to twice the ultimate dry-weather-flow rate can be pumped by the dry-weather-flow pumps, each of which is automatically started and controlled in speed by float-gear operated by the sewage-level in the suction-sump, and by differential gearing operated by changes in difference between water-levels in the suction-sump and in the discharge-culvert. The pumping-lift varies from 44 feet to 58·5 feet, according to the water-levels in the high-level and low-level sewers. When started up each pump has a capacity of 2,000 gallons per minute, which is increased to a maximum of 5,130 gallons per minute as the variable-speed 160-HP. electric motors are controlled through a speed range of from 682 to 841 revolutions per minute. The pumping-output can thus be varied smoothly between 2,000 gallons per minute on one pump to 30,000 gallons per minute with all six pumps, without any sudden changes in discharge, and the output can at all times be matched with the sewer-discharge over the entire range of flow. Manual control is also available when required for cleaning the suction-sump or for other purposes. In this event telephonic communication is used between the shift engineer in charge of the station and the men in the suction-sump.

The six storm-water pumps are driven by vertical diesel engines through gears located vertically over the pumps, but at ground-level. Any five of these sets will deal with the ultimate wet-weather flow independently of the dry-weather-flow pumps, the capacity of each varying from 8,000 to 19,600 gallons per minute. Space between the engine flywheels and gears is available should fluid transmission be found to be desirable, but this introduction is not expected. Heat-exchangers are housed in the annexe, and in them the heat of the jacket-water is transferred to effluent from the main down-shafts. Fuel-oil tanks are situated underground about 40 feet away from the annexe nearer the grit channels to obviate danger from fire or explosion, and all auxiliaries are provided in duplicate with electric or oil-engine drive to guard against failure of the works electric supply.

Solids of 6 inches diameter can be passed by the dry-weather-flow pumps, and 7-inch solids by the storm-water pumps. All pump-discharge mains have venturi tubes incorporated in them, so that each pump duty is known separately as well as the aggregate discharge from all pumps in operation.

Construction of the pump-well was undertaken only after a preliminary cut had been made over the site down to 21·0 O.D., and the depth of the excavation to formation was thus reduced to 58 feet below the working space at the top. This excavation measured 159 feet by 36 feet 6 inches, and was made by two 5-ton cranes operating grabs, one on each side of the hole, and supplemented by a 3-ton crane to handle timbers. Trimming of excavation was done by pneumatic clay-shovels. The first excavation was through 11 feet of gravel and the remaining 47 feet in London clay, runners being driven 18 inches back from the face of the excavation below clay to allow of a clay garland being utilized and drained to a sump. This was successful, and the work below in the clay experienced no difficulty caused by water from the ballast above. Timbering of the top section was by 3-inch runners, 12-inch by 12-inch walings, and framed struts made of pairs of 12-inch by 6-inch timbers with 6-inch distance pieces between them. Three frames of such struts and walings supported the top section, and below that the excavation was taken down with close-spaced 2-inch poling-boards, 12-inch by 12-inch walings and 12-inch by 12-inch struts. Poling-boards were 5 feet long, and this determined the " lifts " of the concrete, in which the reinforcement was made to suit so that walings could be withdrawn up and over the projecting steel while the poling-boards were still held at their bottom ends by the last concrete " lift." Rapid-hardening cement enabled walings to be withdrawn 24 hours after the concrete had been placed, and since 15,000 cubic yards of concrete were involved, as concreting only started in November, 1934, and the building over the well had to be finished and equipped in September, 1935, full advantage was taken of the saving afforded in constructional time.

Despite the fact that excavation was carried out without undue exposure of the clay to the air, the struts were noticed to be punching into the walings, and, in some places, punishing them badly. Because of variation in the quality of the timber and its degree of wetness it was not easy to find suitable test-pieces, but they were eventually found at 67 and 57 feet below ground. At 67 feet a good dry sample showed under test that a load of 2,000 lbs. per square foot was being experienced, and at 57 feet a load of 2,200 lbs. per square foot was recorded where the heaviest indentations showed on the walings. As these loads were within the loads assumed for design, it was

unnecessary to make any change, but it is interesting to note that the low-level sewers, which had been constructed before the pump-well excavation was undertaken, both developed cracks, although they were in London clay to a depth of 57 feet, and in spite of thoroughly good timbering in the pump-well. The nature of these cracks indicated movement of clay towards the pump-well, but as no further trouble showed after they were repaired subsequent to the building of the pump-well, it may be assumed that the movement was arrested by the concrete.

Possible movement of the foundations of the building around the pump-well and foundations of machinery at the edge of the well was anticipated by the driving of 12-inch bored reinforced-concrete piles before excavation of the well was started. These piles were driven over the whole area within 45-degree wedges of the well to avoid surcharge of the well-structure and to support mass-concrete engine-beds 14 feet thick, damping vibration of the diesel engines should their beats happen to synchronize.

Mechanical and Electrical Plant in Main Pumping-Station.

Location.	Plant.	Brief description.
In pump-well.	Six vertical-spindle dry-weather-flow pumps.	15-inch mixed-flow centrifugal type, capacity 2,000 to 5,300 gallons per minute each.
	Six vertical-spindle storm-water-flow pumps.	30-inch mixed-flow centrifugal type, capacity 5,000 to 19,600 gallons per minute each.
	One pump-well drainage-pump set.	3-inch centrifugal type, driven by 2-B.HP. electric motor.
	One motor-driven ventilating fan.	
In main-engine room.	Six electric motors for dry-weather-flow pumps.	Each 160-HP., variable-speed, direct-coupled to pump below.
	Six diesel engines for storm-water pumps.	Each 500-B.HP., 6-cylinder vertical type driving vertical pump-spindle through bevel gears.
	Two cooling-water pump sets.	Both horizontal-spindle 6-inch centrifugal, one direct-coupled to 18-B.HP. electric motor and the other to 18-B.HP. diesel engine.
	Two circulating pumps.	Both horizontal-spindle 4-inch centrifugal, one direct-coupled to 7-B.HP. electric motor and the other to 7-B.HP. diesel engine.
	Six heat-exchangers.	Cooling engine circulating-water by effluent.

Mechanical and Electrical Plant in Main Pumping-Station—continued.

Location.	Plant.	Brief description.
In main - engine room—*continued.*	Two fuel-oil transfer pumps.	One driven by 6-B.HP. electric motor and one by 6-B.HP. diesel engine.
	Main switchboard.	
	One overhead travelling crane.	12-ton, electrically driven.
On first floor over heat-exchangers.	Six daily service fuel-oil tanks.	24 hours' capacity.
In auxiliary-compressor room.	Two auxiliary air-compressor sets.	For starting main engines; one driven by 10-B.HP. electric motor and one by 10-B.HP. diesel engine.
	Two motor-generator sets.	Providing alternating current for recording instruments.
In basement.	Three oil-fired heating boilers.	
Outside in underground chamber.	Three fuel-oil storage tanks.	Aggregate capacity 100 tons.
Machine-shop.	One heavy - duty lathe.	10½-inch centres.
	One heavy - duty lathe.	6½-inch centres.
	One shaping machine.	
	One screwing machine.	
	One sawing machine.	
	One grinding machine.	
	One radial drilling machine.	
	One vertical-pillar drilling machine.	
	One bench drilling machine.	
	One oxy-acetylene cutting and welding outfit.	
	One 6-ton overhead travelling crane.	Electrically driven.
Smiths' shop.	One forge.	4 feet square, with fans.
	One hardening furnace.	(Coal gas.)
	One bar - bending machine, anvils, and swage-block.	
Carpenters' shop.	One circular-sawing machine.	
	One planing machine.	

General Construction.

Approximately half the site occupied by the secondary sedimentation-tanks, the aeration-tanks, and the final separating-tanks had

been excavated under contract "M.1" from the original ground-level of about 35 O.D. to about 20 O.D., but the lower level was covered with semi-fluid silt lying at varying levels and the high level was largely occupied by the old Heston and Isleworth sewage-works. The general system of construction adopted was to excavate the upper 15-foot face from the top, working both it and all other work at the lower level on a north-and-south front starting from the east and withdrawing to the west to leave practically finished work behind. Thus not only the long narrow channels of the aeration-tanks, but also the secondary sedimentation-tanks and the final tanks grew section by section within the radius of the 70-foot jibs of the line of 15-ton steam derricks.

Handling of most of the material, which in this contract, "M.3", approached 1,000,000 cubic yards, was therefore done on standard-gauge track laid with 80-lb. rails, in 15-cubic-yard dump-cars drawn in trains of three or four by steam locomotives, all track being on the high level. Low-level tracks were all of 2-foot gauge, their general use being for the conveyance of concrete, and petrol-locomotives being employed for haulage. Access-ramps had to be maintained not only between the high level and the low level but also down to the general tank formation-levels (6·50 O.D. for secondary tanks, 4·79 O.D. for aeration-tanks, 10·00 O.D. for final tanks). As the work proceeded from stage to stage the layout of tracks and ramps required remodelling and altering to maintain satisfactory working, and several timber bridges were constructed to carry the tracks over road cuttings and across the Duke of Northumberland's river.

For the bulk excavation diesel-driven excavators with capacities of about 1,000 cubic yards each per shift were employed, this being one of the first occasions on which such large internal-combustion-engined excavators have been used. Two of them were in continuous use throughout the bulk-excavation period, and their work was supplemented by three steam excavators, one of 1,200 cubic yards capacity per shift and two of 800 cubic yards; all loaded direct into the 15-ton dump-cars on the high level. Smaller quantities of excavation for foundations, trenches, etc., at the lower level were done by excavators, trenchers, and, in the case of the conical-bottomed final separating-tanks, by grabs operated by the derricks. Steel boxes of 4 cubic yards capacity were used for the excavated material at the low level, being picked up by the derricks and tipped against the face of the bulk excavation, to be reloaded into the 15-ton dump cars.

Good-quality ballast was found in the bulk excavation, and it was stored separately from the remainder to be subsequently re-dug, washed, and screened for concrete aggregates. Two washing and

screening plants were ultimately installed, the larger with a capacity of 900 cubic yards per shift, which is believed to be one of the largest in the country.

Shoring of sides of excavations for tanks was avoided to a great extent by the formation of batters, and clay banks cut as steep as 2 to 1 proved sufficiently stable for the time they were required to stand, whilst in the 36-foot-wide trench for the operating-gallery a face of clay 8 feet high could generally be relied upon to stand with very little or no support. Narrower trenches for culverts and channels were timbered in the usual way. The amount of ground-water encountered was almost negligible.

Ballast-washing and grading plant installed for the ballast recovered by the contractors under contract "M.1" being suitably placed, the concrete batching and mixing plant required under contracts "M.3" and "M.5," was located alongside, so that cranes lifted the aggregates direct into bins over the batcher. Cement from the cement-store was brought in bulk by spiral conveyor to the weigher of the batching and mixing plant. Subsequently another batcher and mixer of the same size was installed at the north end of the site to work in conjunction with the ballast-washing and screening plant there, and the total available concrete-mixing output on the three contracts, "M.3," "M.4," and "M.5," rose to 1,200 cubic yards per shift. It is doubtful whether this maximum output was ever attained, but rates up to 1,000 cubic yards of concrete poured in 1 day were not uncommon. The batching plant was erected immediately south of the south works-road, which was excavated at an early date to provide access for the double track for the concrete-trains at the level of the tops of the tank-walls. These tracks were continued up the east side of the aeration-tanks and round to the secondary sedimentation-tanks, spur roads being laid to bring the concrete within reach of cranes. Concrete-skips were of special design, with a bottom outlet controlled by a lever operated by a man who travelled with the skip when it was lifted by the crane. Each skip contained 2 cubic yards of concrete, and was mounted on a car having two four-wheeled bogies running on 2-foot-gauge track. The bottom outlet extended the whole length of the skip, some skips having clam-type doors, and others a special cylindrical door.

The availability of cranes in sufficient number to save all but essential manual labour manifested itself in shuttering methods. Although pins were cast into concrete foundations or floors to strut the wall-shuttering, other methods, such as strutting from kentledge blocks or skips of clay, were made possible. Wall-shutters were large steel-faced timber panels handled by cranes.

The aeration-tanks involved the construction of about 4 miles of

special curved-section, or "wine-glass," walls, and afforded facilities for the use of special steel formwork. An arrangement was adopted which consisted of three types of light steel carriage on rubber wheels carrying shutters designed to form the three successive lifts of these walls. A set of five carriages of the first type formed a length of 25 feet of the bottom curved lift of four walls together, and when these were moved forward a set of five carriages of the second type was brought up to form the middle lift. They in turn were followed by the set which formed the top curved portion containing the pipe trough. As the top lift was more delicate and required a longer time before its forms could be struck, a double set of forms of the third type was provided. An output of 125 feet of aeration-unit, consisting of four walls, was obtained per week. The footings to walls were completed up to the level of the underside of the floor-slab; the $\frac{3}{4}$-inch asphalt expansion-joint was then laid on the sides of the wall footing, followed by the 9-inch concrete floor over this asphalt. This was the bed upon which the rubber-tired wheels of the steel carriages travelled. The wheels were screwed up clear of the floor when the form was set, and the tops of two carriages in each set were decked over to form stages for the concrete, and for the workmen filling and punning the concrete. Wooden stop-ends were bolted to flanges provided for the purpose at the ends of the shutters. As the bottom lift of the wall is wider at the base than at the top, provision was made to prevent uplift due to the wet concrete by casting pockets in the floor-slab with an iron bar to which holding-down chains were fixed, the pockets being filled later. Complete carriages were lifted by derricks to their new positions after the completion of each unit. The end walls were completed in advance of the long walls of the channels, and the transverse baffle-walls at a later date, chases being left in the side walls to receive them. Construction of the operating-gallery was kept slightly in advance of the tank-walls, of which it forms the south end.

All newly-trimmed clay surfaces were covered at once with a 3-inch sealing coat of 1 : 2 : 4 concrete, and no trouble has been experienced from swelling of the clay.

Excavation-Equipment employed on Contracts "M.3," "M.4," and "M.5."

1 No. 20 Ruston steam navvy.
2 No. 10 ,, ,, navvies.
4 No. 43B diesel-driven ,,
16 No. 4 Ruston navvies, both diesel and petrol driven.
120 $1\frac{1}{2}$-cubic-yard 2-foot-gauge tumbling skips.
24 15-cubic-yard standard-gauge dump cars.

5 standard-gauge steam-locomotives.
25 20-HP. petrol-locomotives.
2 miles of standard-gauge track, 80-lb. rails.
1 mile of 2-foot-gauge track, 30-lb. rails.

The main concreting plant was :—

2 washing and screening plants.
2 batching and mixing plants.
16 10- and 15-ton derrick cranes.
8 10-ton travelling cranes.
80 2-cubic-yard bottom-opening concrete skips.
35 20-HP. petrol-locomotives.
3 miles of 2-foot-gauge track.

It has been estimated that the value of the constructional plant used on these three contracts was about £150,000.

Perry Oaks Sludge-Disposal Works.

Isolation from existing dwelling-houses and unlikelihood of building-development taking place in the immediate vicinity recommended the Perry Oaks site for sludge-disposal, although the low cost of the land (about one-sixth the price per acre of the Mogden land) was an important factor. Although only 7 miles from Mogden, it is no less than 3 miles from the nearest railway-station. The nearest habitable dwelling is an isolated farm 700 yards from the drying beds ; the nearest building-development lies on the Bath road, more than ½ mile to the north of the site.

The total area of land acquired was 250 acres, including 19 acres on the west side of the Longford river and 21 acres between the Longford river and the Duke of Northumberland's river, which, although not of much value as sites for sludge-tips, were of value to the County Council in giving them control over development in that area. The area made available for sludge-treatment and drying is approximately 175 acres, allowing generous margins between it and the public roads.

Since the strata of the site are ballast and light soils overlying the London clay, objection to the proposals was taken by the Metropolitan Water Board on the grounds that sub-soil water finding its way into the Thames, from which the Board draws a supply, would be affected ; agreement was reached that a clay-puddle trench should be constructed surrounding the available site, and taken through the ballast to key well into the London clay. This puddle wall encloses the area of 175 acres referred to above. A branch sewer is provided to convey all rain-water falling inside the area, together with drainage from the drying-areas, to the main sewer in the Bath road, and thence to Mogden for full treatment as sewage.

The first contract let was for the construction of the puddle wall ; the minimum width specified was 2 feet, but the actual width as

executed was 2 feet 3 inches. The puddle used was clay from tunnel-excavation on one of the sewer-contracts, the clay being specially tipped on the Perry Oaks site for this purpose under the conditions of contract.

The maximum depth of ballast encountered was 25 feet, the minimum 12 feet, and the total length of puddle trench executed under the contract was 3,864 yards. Most of the land had been an old orchard, and about 50 acres had to be cleared of trees and roots, under this contract, before work was started on the site.

On the completion of this preliminary work the way was clear for the letting of a contract for the main constructional programme on the site, and, unlike the Mogden work, it was practicable to advertise it *in toto* as one contract. The work comprised secondary sludge-digestion plant, sludge-drying beds, and sludge-pumping station, together with roads, tracks, and buildings.

Secondary Sludge-Digestion Tanks.

The tank-capacity provided is 2½ million cubic feet, making with the capacity of 1·66 million cubic feet at Mogden an aggregate total of 4·16 million cubic feet, or nearly 3·3 cubic feet per person for a population of 1,250,000.

Ten circular tanks (Fig. 42, Plate 3), each 100 feet in diameter, with a side-water depth of 30 feet 5 inches and conical floors laid at 1 in 12 were adopted, the floors being laid on a 3-inch sealing coat of concrete as at Mogden, because here too the foundations are in London clay.

Delivery-pipes enter the tanks at one point only, where a vertical leg is erected against the wall with outlets at several levels to obviate splashing when the tank is nearly empty. Suction-pipes are laid on the floors of the tanks, and dip at the centres to small sumps. Supernatant-water draw-off pipes 9 inches in diameter are provided at three levels, 3 feet, 6 feet, and 9 feet respectively below top sludge-level. They are concentrated and controlled by valves at central control-chambers, each of which serves four tanks. All pipes passing through tank-walls are accommodated in cast-iron sleeves, double-socketed, and provided with puddle-flanges as in the case of the primary sludge-tanks at Mogden.

The depth of excavation for these tanks was about 26 feet, and it was preferable to remove the 12-foot depth of ballast over the whole area rather than to leave small dumplings between tanks. To control the water in the ballast, steel sheet-piling was driven to enclose the whole site of the tanks, being located sufficiently far from the tanks to enable ballast left inside to support the piling, and to

obviate the use of shoring. On completion of the work, the piling was withdrawn. The ballast excavated was conveyed to the gravel-washing and screening plant, and thence for use as medium in the drying beds. In the construction of the tanks, a circular rail-track was laid on the floor-sealing coat, carrying a specially-made steel framework mounted on wheels, from which sections of steel shuttering were hung, so that the moving and setting of the shuttering for each lift was simplified. Each lift of the walls was limited to 4 feet in height, being poured in alternate bays not exceeding 60 feet in length, and the floors were completed after the walls were finished. Concrete was mixed by batcher at the central mixing-plant, and was delivered to the place of deposit by pumping.

Sludge-Drying Beds.

The area of beds provided is 50½ acres, divided into sixty-two plots each 300 feet long by 120 feet wide (except in five cases where site-limitations have slightly modified the dimensions). They are arranged in one large block, containing two lines of sixteen beds and two lines of fifteen beds, with the length of the beds at right-angles to the long axis of the block. Two lanes, accommodating a light-railway track, divide the lines of beds, so that each track gives access to beds on left and right. The block is also divided by one roadway running at right angles to the light-railway track, while another road runs completely round the site.

The light-railway track is looped at each end of the block and a triangle is provided to facilitate shunting, a locomotive-shed being built on a siding off the main line. The track is continued across the Duke of Northumberland's river, on a steel-girder timber-decked bridge, to give access to tip-sites. Roadways and lanes are formed in concrete. The division and external walls of the beds are also formed in concrete, and are carried 2 feet above medium level. All walls are 6 inches thick with footings 2 feet or 2 feet 6 inches wide, there being four types of wall, three differing only in the footings owing to slight differences of formation-level of groups of beds. The long division-walls between beds are required as gangways, and are therefore widened out to 18 inches at the top. The approximate length of wall is 6¼ miles. This mileage justified the use of steel formwork, and the amount of repetition no doubt reduced the price considerably. Ordinary earth banks would have wasted 9 per cent. of the area, whereas the loss in effective area with concrete walls is less than 1 per cent., and the cost of keeping down weeds on over 6 miles of earth bank is entirely avoided. Concrete walls are tidier, cleaner, and of better appearance.

The fall across the original ground-surface over the whole area was

less than 4 feet in 600 yards. Since a difference of 1 inch in level made a difference in excavation of about 8,000 cubic yards, care had to be taken in fixing the bed-levels, and the excavation was reduced to a minimum by grouping the beds at four different levels, the highest being 18 inches above the lowest. Turf and top soil were stripped off 6 inches deep, even where filling was required. Excavation was computed at 68,000 cubic yards, and refilling at 13,500 cubic yards, exclusive of excavation for drainage-pipes and wall-footings. General excavation was carried out by scrapers drawn by tractors, and spoil was used to make embankments around the site.

The beds are underdrained with 3-inch porous concrete pipes, laid in herring-bone pattern at about 12-foot 6-inch centres, connecting to a main open-jointed stoneware pipe. This pipe runs parallel to a division-wall, and picks up the porous pipes from two beds. Supernatant-water draw-offs are built in the division-walls, each serving two beds, and directly connected to a stoneware drain. A main intercepting drain is laid down the lane between lines of beds to intercept the stoneware drains, and finally all the drainage is discharged to the branch sewer. The controlling factor in design was the amount of rainfall expected, rather than the amount of sludge-drainage.

Medium for the beds consists of a 1-foot 6-inch depth of gravel in three layers, $2\frac{1}{2}$-inch to $1\frac{1}{2}$-inch gauge, $1\frac{1}{2}$-inch to $\frac{3}{4}$-inch gauge, $\frac{3}{4}$-inch to $\frac{1}{4}$-inch gauge, and a top dressing of sand of $\frac{1}{4}$-inch gauge downwards.

Sludge is delivered to the beds through a 12-inch pumping-main, with 9-inch branches running down the centre of each bed; each branch is controlled by a sluice-valve adjacent to the lane, and the sludge is discharged from three vertical stand-pipes equally spaced down the centre-line of the bed. Special removable bellmouth tops to these upstands are fitted, for convenience in sluing the light-railway track during the operation of emptying a bed, and the height of the bellmouths can be regulated to secure equalization of discharges. Although the maximum distance which sludge has to flow in a bed is 78 feet, there is a slight tendency for the sludge to form cones with each inlet as an apex, and as these cones intersect, the wettest sludge reaches the walls half-way between outlets. The supernatant-water draw-off chambers have therefore been located at these points, there being four on each side of each bed.

An opening is formed in the concrete wall of each bed next the lane, at the corner of the bed, to give access for sludge-lifting. This opening is normally closed by two sets of 3-inch stop planks, dropped into groves at the sides of the opening.

Light-railway track is composed of 25-lb. rails, embedded in the concrete of the lane, except opposite each opening, where the concrete is sunk so that two standard lengths of rail are exposed. One of these lengths is removable and a standard turnout, either left- or right-hand, can be dropped into its place and fished-up, to give access to the appropriate bed. Jubilee wagons are provided, together with two 27/32-HP. diesel-engined locomotives.

On the west side of the site a sectional timber building has been erected on a concrete floor, as a men's shelter. Water is laid on to lavatory basins, and a central fireplace provided for warming food and drying clothes.

Pumping-Station.

The pumping-station building also houses the manager's office and men's canteen, with lavatory accommodation and spray-baths. It is a single-storey building, with two galleries in the pump-room to accommodate the fuel-oil storage tank, batteries, and cold-water cistern. The dry pump-well is 33 feet by 21 feet by 15 feet deep, and is founded on clay, whereas the building itself is founded on gravel, and the two structures are therefore quite independent, a sliding joint being formed by cork and bitumen. A hand-operated 3-ton travelling crane commands the pump-bay, in which are two identical horizontal duplex pumps belt-driven by diesel-engines. The pumps are capable of discharging 500 gallons per minute against a pressure of 100 lbs. per square inch, and the ratio of the drive can be altered if required, so that the output can be made 1,000 gallons per minute against a head of 50 lbs. per square inch. The engines can develop 92 B.HP. on oil fuel, and are capable of conversion to run on sludge-gas if desired.

Although it is not proposed at present to collect gas from the secondary digestion-tanks, two tanks have been constructed with a resting-corbel to carry a gasholder-cover, should operating experience show the collection of gas to be justified. In this event, these two tanks can be treated as reception-tanks, and after a few days the sludge can be pumped to the other tanks for storage and complete digestion. Laying of the rising main as a closed circuit and the provision of a double suction pipe enable this to be done, as well as permitting the mixing of sludge by pumping from one tank to another should this be desirable. The rising mains from Mogden are also cross-connected to the rising main from the above pumps, so that the tanks can be by-passed if necessary and sludge delivered direct from Mogden on to the drying-beds. The delivery and suction mains are all 12-inch cast-iron pipes.

Gravel-Pit.

As over 22,000 cubic yards of concrete were placed, and over 122,000 cubic yards of various grades of sand and gravel were required for medium for the drying-beds, tenderers were given the option of opening and working a gravel-pit on the site, instead of buying sand and gravel for concrete-aggregate and drying-bed media, and five out of the six lowest tenderers based their tenders on working their own gravel-pit. Alternative prices inserted in the bill for gravel from outside sources suggest that a saving of £17,500 was thus effected, and in addition the gravel-pit was made available as a tip for dried sludge. The pit was worked as a wet pit, with floating barges carrying a suction-dredger. Ballast was screened and washed on vibrating screens near the pit, and the power and lighting required for all temporary plant was produced by diesel-engines driving generators.

Cost.

In 1930 the cost was estimated for Parliamentary purposes as £5,250,000, made up as follows :—

	£	£
Purchase of land and easements		125,000
Works and buildings	1,287,000	
Sewers	3,238,000	
		4,525,000
Purchase of existing sewage-works		600,000
		£5,250,000

Inclusive of all the overhead charges which the Parliamentary estimate takes into account, the sewers are expected to cost about £3,150,000 as compared with their estimated cost of £3,238,000; a saving of about £88,000 is thus anticipated.

Under " works and buildings " was included all work at Mogden and at Perry Oaks, the gross expenditure on which is not known at the time of writing (October, 1936), but in February, 1936, estimates prepared for the County Council showed the probable aggregate cost to be £2,006,000. Although this is an apparent excess of £719,000 over the estimates, two facts have to be considered. Firstly, the Parliamentary estimate was framed for works to serve 750,000 persons, whereas as finally constructed they can serve 1,250,000. Secondly, quite apart from the additional population served, the work carried out at Mogden and Perry Oaks, particularly the former, is incomparably greater than that foreshadowed by the 1930 estimates. Among the many items of additional work and serious obstacles entirely unforeseen in 1930, so far as the 1935 programme was concerned, were the demolition

of the Heston and Isleworth works, and the construction of the new works on the same site at a lower level; the construction and maintenance of temporary works for treatment of both sewage and sludge from Heston and Isleworth, serving a population of about 90,000 persons; the diversion of ½ mile of the Duke's river to its new line through the middle of the works, with its attendant bridges, culverts, new public footpath, and approach-road to the works; the severance of the site caused by the river, public footpath and approach-road constructed at a high level; the diversion of cables of two authorities; the moving of the ballast-washing, screening, and storage plant to another site; and the increase in the power- and compressor-house from half to full size.

In the Parliamentary estimates of cost, figures were given for the expenditure expected to be necessary, subsequent to the completion of the 1935 programme, to provide extensions to plant to serve the increasing population expected over a period of years. The sum forecast for works of the size actually constructed was £1,787,000, and on this basis the estimated aggregate gross cost of the whole scheme, works and sewers, becomes £5,025,000. The expected total cost of £5,156,000 will therefore exceed these estimates by about £131,000, or 2½ per cent.

The unemployment-relief grant to be made by the Government was conditional, as previously stated, and among the stipulations was the obligation to complete the works by the 1st October, 1935, and thus to construct work to a value of £4,525,000 within the 4-year period allowed. The value of work actually carried out on the 30th September, 1935, was £4,775,000.

In spite of the purification-plant being 67 per cent. larger than originally proposed, work was sufficiently advanced on the 14th December, 1935, for the first diversion of sewage to be made, or only about 10 weeks after the appointed date. After a trial run of 6 weeks the main programme of connexions from local authorities' sewage-works in different parts of the county was steadily undertaken, completion being effected in May, 1936; twenty-eight old sewage-works have thus been thrown out of commission.

In pursuance of the Act the expenses incurred will be defrayed by special rate levied on the Drainage District. It was expected that the expenditure would necessitate a rate-charge rising through the constructional period to 6·5*d.* in the pound for the first year of operation. In spite of the heavy extra commitments due to larger works, the rate actually levied for the year 1936–7 was only 6·0*d.* in the pound, and it is expected that future charges will be less than Parliamentary forecasts, owing to the greatly-increased population contributing.

Most of the constituent local authorities will effect actual savings by this rate, which is in substitution of their old rate for sewage-treatment and disposal; many of them would have been obliged to face heavy expenditure on enlargement of their old works, with a resulting increase in their old rate-charges, and on this account large savings may be expected both on construction of works and by reason of cheaper development of local sewerage. Several of the districts are already paying lower rates than they paid before the inception of the scheme. Roughly 1,000 acres of land formerly used as sewage-works sites have now become available for other purposes, so that the community will gain not only by appreciation in value of the neighbouring land but by realizing in some form the value of the 1,000 acres, the majority of which is located in urbanized districts.

All work was carried out by contract, and the contractors are named in Appendix III.

Resident Engineers were responsible during the early months of construction to Mr. H. R. M. Macmillan, V.D., M. Inst. C.E., who was succeeded on his death by Mr. R. A. T. Anderson as Chief Administrative Engineer. The Resident Engineers were Messrs. F. W. Ireland and J. H. Mair, B.Sc., MM. Inst. C.E.; W. E. Laird Adams, L. B. Aylen, F. J. Crabb, B. Eng., H. H. Hunt, B. E. Ireland, A. F. St. J. Kinsey,[1] and A. G. Wilbond, B.E., Assoc. MM. Inst. C.E.; J. A. Jameson and A. Pinkham.[1]

Each Resident Engineer had at least one Assistant Engineer to help him.

Clerks of Works and Inspectors up to a maximum number of sixty-three were employed, the average number during the 48 months of active construction being forty-seven. The cost of their services varied with the character of the work, the speed of construction, and the degree in which the construction approached the average during the first and last few months. Subject to these uncertainties, however, the Table on p. 557 may be of interest, representing as it does the percentage which the total wages and allowances paid to Clerks of Works and Inspectors on the site of the constructional work bears to the total amount finally paid to the Contractor.

All architectural work was designed and carried out by Messrs. Page L. Dickinson and F. Norman.

General inspection of manufactured materials and machinery was carried out by Mr. J. Haggie Patterson, Assoc. M. Inst. C.E., except for cement-testing and concrete-pipe inspection by Messrs. Riley, Harbord and Law, and examination of main engines, compressors and generators by Lloyd's Register of Shipping.

[1] Since deceased.

Contract No.	Percentage cost of Clerks of Works on cost of work.	Contract No.	Percentage cost of Clerks of Works on cost of work.	Contract No.	Percentage cost of Clerks of Works on cost of work.
S.2	1·38	S.14	2·30	S.26	1·64*
S.3	1·67	S.15	1·48	S.27	1·90
S.4	2·09	S.16	2·02	S.28	2·07
S.5	1·32	S.17	1·80	S.29	3·20
S.6	0·94	S.18	2·18	M.2	1·21
S.7	1·93	S.19	2·12	M.3	0·63*
S.8	1·82	S.20	2·73*	M.4	0·97*
S.9	2·06	S.21	1·40	M.5	0·54*
S.10	1·90	S.22	1·17	P.1	0·85
S.11	1·88	S.23	2·61	P.2	1·22*
S.12	1·53	S.24	2·13		
S.13	1·87	S.25	1·71	Total Scheme	1·40*

* Final figures not yet available.

In addition to manufacturers' drawings supplied in connexion with machinery and equipment, about 1,900 drawings were prepared for the execution of the work and about 18,000 prints supplied. Roughly 2,800 easement-plans were required.

Throughout the whole period of design and construction the loyalty of all concerned was a marked feature of the work. Contractors and their staffs went out of their way on many occasions to put the County Council's interests before their own, and it would be difficult to exaggerate, or speak too highly of, the unselfish keenness of the many members of the Engineers' staff, who toiled long hours for years on end to give of their best.

The Author is greatly indebted to the following members of his staff, who were closely identified with the work, having all been responsible for design and construction, and whose notes, criticism and other help in the compilation of this Paper have been invaluable :—Messrs. P. G. Smales, Assoc. M. Inst. C.E., Chief of Staff; C. B. Townend, B.Sc., M. Inst. C.E., Chief Assistant Engineer on the scheme, now Engineer-in-Charge for the County Council; R. A. T. Anderson, Chief Administrative Engineer; W. Parker, M. Inst. C.E.; C. D. C. Braine, B.Sc., R. A. Elliott, B.Sc., Charles Hogg, B.Sc., H. D. Manning, B.Sc., D. C. McCormick, C. A. Stewart, M. W. Summers, and G. W. Wilkinson, Assoc. MM. Inst. C.E.

The Paper is accompanied by fifty-two sheets of drawings and twelve photographs, from some of which Plates 1, 2, and 3, the Figures in the text, and the three half-tone page-plates have been prepared, and by the following three Appendixes.

APPENDIX

SEWERS.

Contract.		Internal diameter.	Approximate				
			Open Trench.				
No.	Description.		Cast-iron pipes.	Concrete tubes.	Brick and concrete.	Cast-iron pipes.	Concrete tubes.
		ft. ins.					
S.2	Bath Road main . .	9 0			281		
S.3	Crane Valley main .	6 6			499		
		6 9					
		7 0			51		
S.4	Harrow and Pinner branches.	1 9		165			
		2 0		1,605			
		3 0		567			
		3 6		768			333
		4 0			1,344		
		4 6			1,293		
S.5	Wembley branch . .	4 6			535		
S.6	Wealdstone and Stanmore branches.	1 9	99	176			
		2 6		115			1,970
		3 0		1,046			1,236
		3 3					1,134
S.7	Brent Valley main (upper section).	6 6			1,175		
		7 0			269		
		7 9			1,298		
S.8	Hendon branch . .	4 6					
		5 6			13		
		6 0			5		
			99	4,442	6,763	—	4,673

I.

length : yards.							
Tunnel.							
Brick.	Brick and concrete.	Concrete segments.	C.I. segments hand driven.	C.I. segments shield driven.	Total	Cost : £.	Remarks.
22		4,414	365*	532*	5,614	281,234	* Includes 760 yards in compressed air. One railway crossing.
		2,667 1,144 4,110		545*	9,016	299,418	* Includes 218 yards in compressed air. One railway crossing, one canal crossing.
867			170		7,112	81,412	One railway crossing, one canal crossing.
2,925	7		201		3,668	58,065	One railway crossing, one river crossing.
					5,776	52,844	
		2,175* 1,786§ 131	459† 90	327‡	7,710	237,032	* Includes 9 yards in compressed air. † Includes 67 yards in compressed air. ‡ Includes 282 yards in compressed air. § Includes 125 yards in compressed air. Six railway crossings, one river crossing, one canal crossing.
2,677 849 384			30 149*		4,107	82,105	* Includes 25 yards in compressed air. One railway crossing, two river crossings.
7,724	7	16,427	1,464	1,404	43,003	1,092,110	Carried Forward.

APPENDIX I—

SEWERS—

Contract.		Internal diameter.	Approximate				
			Open trench.				
No.	Description.		Cast-iron pipes.	Concrete tubes.	Brick and concrete.	Cast-iron pipes.	Concrete tubes.
		ft. ins.	99	4,442	6,763	—	4,673
S.9	Bath Road main (upper section).	4 6			1,702		
		5 0					
S.10	Colne Valley main and Cowley branch.	2 0	3,400				
		2 9	422				
		3 4	2,423				
		3 10	1,113				
		4 0	680				
S.11	Hendon branch connecting sewers.	1 9	50			44	
		2 0	92				17
		2 3	200	223		75	
		3 3					159
		3 4	372				
S.12	Uxbridge and Ruislip branches.	0 9	349			76	
		1 0	2,101			341	
		1 9	100			68	
		2 9		502			528
		3 9		496			1,399
S.13	Northolt, Greenford and North Ealing branches.	1 0	155				
		1 6	742	994			
		1 9	35	547			
		2 6		959			70
		3 3		400			
S.14	Eastern low-level (upper section).	3 0					
		4 0					
S.15	Brent Valley main (central section).	1 3		99*			
		10 0			1,769		
		10 3			481		
		10 6			1,338		
			12,333	8,662	12,053	604	6,846

continued.

continued.

length: yards.						Cost: £.	Remarks.
Tunnel.					Total.		
Brick.	Brick and concrete.	Concrete segments.	C.I. segments hand driven.	C.I. segments shield driven.			
7,724	7	16,427	1,464	1,404	43,003	1,092,110	Brought Forward.
		1,687 1,550			4,939	109,948	One river crossing.
				47 119 152	8,356	115,493	Three railway crossings, eight river crossings, two canal crossings.
					1,232	17,311	Two river crossings.
					5,960	47,524	One railway crossing, one canal crossing, two river crossings.
					3,902	19,587	One canal crossing.
			328	759* 932†	2,019	81,560	* All in compressed air. † All in compressed air.
307 380			161 397	233†	5,165	208,873	* Stoneware pipes in concrete surround. † Includes 108 yards in compressed air. Three railway crossings, one river crossing, one canal crossing.
8,411	7	19,664	2,350	3,646	74,576	1,692,406	Carried Forward.

APPENDIX I—

SEWERS—

Contract. No.	Contract. Description.	Internal diameter. ft.	Internal diameter. ins.	Approximate: Open trench. Cast-iron pipes.	Open trench. Concrete tubes.	Open trench. Brick and concrete.	Cast-iron pipes.	Concrete tubes.
				12,333	8,662	12,053	604	6,846
S.16	Western low-level .	4	0	66				
		4	3					
		4	6					
		5	0					
		5	3					
		5	6					
		6	0					
		6	6					
		6	9					
		7	0					
S.17	Effluent-conduits .	6	0					
		11	0					
S.18	Ruislip branch (lower section).	4	6					
		5	3					
S.19	Eastern low-level (central section).	2	0					
		4	0					
		4	6					
		4	9					
S.20	Teddington branch .	0	7	90				
		0	9	211				
		1	0	884				
		1	3	84				
		2	6					577
		3	0					3,285
		3	9					459
S.21	Brent Valley main (lower section).	2	6	103				
		9	0			4		
		10	6			95		
		10	9			918		
		12	9			377		
S.22	Effluent-conduits (land section).	11	0					
				13,771	8,662	13,447	604	11,167

continued.

continued.

length: yards. Tunnel. Brick.	Brick and concrete.	Concrete segments.	C.I. segments hand driven.	C.I. segments shield driven.	Total.	Cost: £.	Remarks.
8,411	7	19,664	2,350	3,646	74,576	1,692,406	Brought Forward.
		1,514 2,488 2,158 295 1,118 1,967 951 2,214 228 1,465			14,464	368,808	Two railway crossings, three river crossings, two aqueduct crossings.
				493 790	1,283	102,994	Two crossings under river Thames.
		2,430 762			3,192	55,406	One railway crossing.
		52	101 1,620 1,614		3,387	81,879	One railway crossing, one river crossing, one canal crossing.
				1,020*	6,610	110,200†	Two railway crossings. * Includes 941 yards in compressed air. † Final cost not yet available.
		44 153	121	42* 195*	2,052	129,322	* All in compressed air. One railway crossing, one river crossing.
				1,955	1,955	106,107	Two river crossings.
8,411	7	37,503	5,806	8,141	107,519	2,647,122	Carried Forward.

APPENDIX I—

SEWERS—

Contract. No.	Contract. Description.	Internal diameter.	Approximate — Open trench. Cast-iron pipes.	Open trench. Concrete tubes.	Open trench. Brick and concrete.	Cast-iron pipes.	Concrete tubes.
		ft. ins.	13,771	8,662	13,447	604	11,167
S.23	Eastern low-level (lower section).	4 0					
		4 9					
		5 0					
S.24	Sunbury and Feltham branches.	0 9	9				
		1 9	517				94
		2 6		467			1,402
		2 9					979
		3 0					780
S.25	Hanwell and South Ealing branches.	0 6	136				
		0 9	309				
		1 0	205			63	
		1 4	71			240	
		1 6	34				
		1 8	205			63	
		2 0	95				
		2 3	71			240	
		2 6	60			24	
		3 0		222			24
		3 7					
		3 9		415			588
		4 7					
		5 9					
S.26	Brent Valley main (lower section).	10 6					
S.27	Isleworth branches.	0 9					173*
		1 0	209				1,059*
		1 3					773
S.28	Sludge-main and Perry Oaks branches	0 6	420				
		1 0	9,920			12,400	
		1 6	1,104				
		2 6	38				
S.29	Western low-level (upper section).	3 0	644				
			27,818	9,766	13,447	13,634	17,039
				51,031			

continued.

continued.

lengths: yards.						Cost: £.	Remarks.
Tunnel.					Total.		
Brick.	Brick and concrete.	Concrete segments.	C.I. segments hand driven.	C.I. segments shield driven.			
8,411	7	37,503	5,806	8,141	107,519	2,647,122	Brought Forward.
		734 1,583 749			3,066	65,025	One railway crossing.
					4,248	54,384	One railway crossing.
			19 124 36 50		3,294	28,672	One railway crossing, one canal crossing.
		1,671			1,671	86,000*	* Final cost not yet available.
					2,214	23,969	* Stoneware pipes in concrete surround. One railway crossing.
					23,882	54,500*	* Final cost not yet available.
					644	8,192	
8,411	7	42,240	6,035	8,141	—	—	
95,507					146,538	2,967,864	

APPENDIX II.

Sewers Classified According to Size.

Internal diameter.		Approximate length: yards.		
feet.	inches.	Open trench.	Tunnel.	Total.
0	6	556	—	556
0	7	90	—	90
0	9	878	249	1,127
1	0	13,474	13,863	27,337*
1	3	183	773	956
1	4	71	240	311
1	6	2,874	—	2,874
1	8	205	63	268
1	9	1,689	206	1,895
2	0	5,192	165	5,357
2	3	494	315	809
2	6	1,742	5,063	6,805
2	9	924	1,626	2,550
3	0	2,479	6,103	8,582
3	3	400	1,293	1,693
3	4	2,795	152	2,947
3	6	768	333	1,101
3	7	—	124	124
3	9	911	2,446	3,357
3	10	1,113	—	1,113
4	0	2,090	5,128	7,218
4	3	—	2,488	2,488
4	6	3,530	14,766	18,296
4	7	—	36	36
4	9	—	1,635	1,635
5	0	—	2,594	2,594
5	3	—	1,880	1,880
5	6	13	2,816	2,829
5	9	—	50	50
6	0	5	1,977	1,982
6	6	1,674	7,842	9,516
6	9	—	1,372	1,372
7	0	320	7,996	8,316
7	9	1,298	131	1,429
9	0	285	5,377	5,662
10	0	1,769	161	1,930
10	3	481	937	1,418
10	6	1,433	2,325	3,758
10	9	918	42	960
11	0	—	2,745	2,745
12	9	377	195	572
		51,031	95,507	146,538

* Includes twin sludge-main to Perry Oaks.

APPENDIX III.

CONTRACTS.

Contract No.	Subject.	Contractor.
	MAIN SEWERAGE.	
S.1	Boreholes.	C. Isler & Co., Ltd.
S.2	Bath Road main sewer.	Sir Robert McAlpine & Sons (London), Ltd.
S.3	Crane Valley main sewer.	Sir Robert McAlpine & Sons (London), Ltd.
S.4	Harrow and Pinner branch sewers.	Tarmac, Ltd.
S.5	Wembley branch sewer.	A. Waddington & Son.
S.6	Wealdstone and Stanmore branch sewers.	Wm. Moss & Sons, Ltd.
S.7	Brent Valley main sewer—upper section.	Sir Robert McAlpine & Sons (London), Ltd.
S.8	Hendon branch sewer.	A. Waddington & Son.
S.9	Bath Road main sewer—upper section.	Sir Robert McAlpine & Sons (London), Ltd.
S.10	Colne Valley main and Cowley branch sewers.	Howard Farrow, Ltd.
S.11	Connecting sewers—Hendon branch.	W. and C. French, Ltd.
S.12	Uxbridge and Ruislip branch sewers.	W. and C. French, Ltd.
S.13	Northolt, Greenford and N. Ealing branch sewers.	Bentley and Wardman.
S.14	Eastern low-level sewer—upper section.	Cleveland Bridge and Engineering Co., Ltd.
S.15	Brent Valley main sewer—central section.	Paterson and Dickinson, Ltd.
S.16	Western low-level sewer—lower section.	Sir Robert McAlpine & Sons (London), Ltd.
S.17	Effluent-conduits—river section.	Cleveland Bridge and Engineering Co., Ltd.
S.18	Ruislip branch sewer—lower section.	Sir Robert McAlpine & Sons (London), Ltd.
S.19	Eastern low-level sewer—central section.	Sir Robert McAlpine & Sons (London), Ltd.
S.20	Teddington branch sewer.	Cleveland Bridge and Engineering Co., Ltd.
S.21	Brent Valley main sewer—lower section.	Sir Robert McAlpine & Sons (London), Ltd.
S.22	Effluent-conduits—land section.	Sir Robert McAlpine & Sons (London), Ltd.
S.23	Eastern low-level sewer—lower section.	Sir Robert McAlpine & Sons (London), Ltd.

APPENDIX III—*continued.*

CONTRACTS—*continued.*

Contract No.	Subject.	Contractor.
	MAIN SEWERAGE—*continued.*	
S.24	Sunbury and Feltham branch sewers.	Wm. Moss & Sons, Ltd.
S.25	Hanwell and S. Ealing branch sewers.	W. and C. French, Ltd.
S.26	Brent Valley main sewer—lower section.	Sir Robert McAlpine & Sons (London), Ltd.
S.27	Isleworth branch sewers.	Howard Farrow, Ltd.
S.28	Sludge-main and Perry Oaks branch sewers.	W. and C. French, Ltd.
S.29	Western low-level sewer—Upper section.	Thos. Bugbird & Son, Ltd.
	MOGDEN PURIFICATION-WORKS.	
M.1	Excavation.	Ham River Grit Co., Ltd.
M.2	Storm-water tanks, etc.	W. and C. French, Ltd.
M.3	Secondary sedimentation, aeration and final separating tanks, etc.	Sir Robert McAlpine & Sons (London), Ltd.
M.4	Grit-channels, primary sedimentation-tanks, pumping-station, workshops, administration building, etc.	Sir Robert McAlpine & Sons (London), Ltd.
M.4c	Main sewage-pumping machinery.	Worthington-Simpson, Ltd.
M.5	Sludge-digestion tanks, power and compressor house, etc.	Sir Robert McAlpine & Sons (London), Ltd.
M.5c	Air-compressing machinery and electrical equipment.	Harland and Wolff, Ltd.
	PERRY OAKS SLUDGE-DISPOSAL WORKS.	
P.1	Puddle wall.	Howard Farrow, Ltd.
P.2	Sludge-digestion tanks, drying beds, etc.	Edmund Nuttall, Sons & Co., and John Mowlem & Co. (Joint), Ltd.

Discussion.

The AUTHOR showed a number of lantern-slides illustrating the vorks described in his Paper. The Author.

Mr. J. D. WATSON, Past-President, said that until the Royal Comnission on Sewage Disposal issued their Fifth Report in 1908, the Local Government Board held that, without land-irrigation, sewage ould not be efficiently purified. Since that time, however, coninuous progress had been made, resulting from assiduous study and uccessful experiments on the part of engineers, biologists, and hemists. England had admittedly taken the lead, but America, Germany and other countries, even as far away as Japan, had taken heir share in the work of developing bio-aeration and many other hases of modern treatment. The activated-sludge process which Arden and Lockett had discovered in 1913 had been invaluable, specially when sites were found to be too limited in area to accomnodate even the percolating type of bacteria-bed. He could not peak too highly of the progress made in America, and particularly n Chicago and New York. Mr. Watson.

The Mogden installation was a modern expression of many eatures of sewage-purification. Nearly all of them had been ried and tested by local authorities and makers of machinery, t considerable cost to themselves. One of the interesting features f the kind of work in question was its international character; nany foreign engineers had come to England to learn something of ecent discoveries and to see examples of efficient installations. They had been made conscious of the fact that pioneer work was ostly. The Birmingham sewage-works had begun to build bacteriaeds immediately after they had bought and drained a great area f land to extend their sewage-farm, and more recently Sheffield ad made an even greater sacrifice; they had given up contacteds and had begun to construct bio-aeration tanks before they had pent all the money the Ministry had sanctioned to build them. So much pioneer work had been done in the past, and so many uthorities had borne their share of its cost, that it would be imossible to acknowledge the work of them all; but without doubt he county of Middlesex owed a heavy debt of gratitude to them or having made it possible, by their past endeavours, for Middlesex o design and carry out such an important work in such a short time.

Some of the outstanding features of the Middlesex scheme were:

(1) By the unification of twenty-seven small sewage-works into

Mr. Watson.

one large works, the County Council had made provision for the purification-process to be effected under the direction of their very able engineer.

(2) The provision for receiving surface- and flood-waters into intercepting sewers.

(3) The more effective use made of large storm-water tanks.

(4) The employment of activated-sludge methods of anaerobic treatment for the purification of liquid sewage.

(5) The complete anaerobic digestion of sludge, primarily to secure freedom from smell and secondarily to appropriate the available gas for power and heat.

(6) The use of multiple outlets at the end of the effluent-conduit in order to facilitate intermixture of the purified effluent with the tidal waters of the Thames.

(7) The carrying out of the complete isolation of sludge-drying beds from adjacent land by sinking a puddle-wall through perviou clay and keying it into the London clay. That was probably unique and was a wise precaution in view of the fact that it might well b dangerous if soakage were to find its way into potable water.

All engineers realized that work comparable with that which th Author had described was bound, of necessity, to be team-work and it was not only the duty of those concerned but their very grea pleasure to acknowledge the invaluable help which they had receive from both their indoor and outdoor staff. They would also acknow ledge with gratitude the able and efficient manner in which th Contractors and Sub-Contractors had tackled their work; by thei determination and vigour they had helped materially to complet the scheme within the time-limit laid down by statute. Th encouragement and help which had been received from the Drainag Committee of the County Council, presided over by Sir Willian Prescott, C.B.E., M. Inst. C.E., and the guidance which had bee received from the County Clerk, Mr. C. W. Radcliffe, M.A., in th many instances where legal difficulties had obtruded themselves had all contributed to the early completion of the work.

Sir George Humphreys.

Sir George Humphreys, Past-President, remarked that the Pape would, he was sure, afford much food for thought to all engineer connected with sewage, and more particularly to those whose practic had led them to study drainage questions in and around Londo The drainage problems of London were many and vast. He looke upon the installation which had been described as really the outcon of the first co-ordinated plan for London sewage, which had bee laid down by Sir Joseph Bazalgette, C.B., Past-President Inst. C.E about the year 1855. With the spread of building around Londo further facilities had been required, and he was afraid that they ha

Sir George Humphreys.

een given in the past by what might be termed very parochial nethods. The principal drainage authority for London, the London ounty Council, had for many years been solicited by outlying istricts to give sewage-facilities, and the London County Council ad done so as far as possible, selling, in fact, such sewage-facilities s it had been able to give to recipients who had been only too glad o pay the price.

The unsuitability of a local government area forming the area for ertain public services was well known; it was more especially the ase where drainage questions were concerned and where the natural oundary was clearly the boundary of the watershed. Those diffi- ulties were so strongly emphasized just before the War that a Royal ommission had been appointed in 1921 to inquire and report on hat, if any, alterations were needed to the local government adminis- rative County of London and surrounding districts with a view to ecuring greater efficiency and economy in the administration of ocal government services. Drainage had formed a very important art of that inquiry, and it might be said that on the question of rainage there had been a large measure of unanimity, and that the aajority and two minority reports of the Commission had all recog- ized the necessity of a comprehensive scheme for dealing with the rainage of London. Arising out of those findings, the Ministry of Iealth had undertaken an inquiry into the matter, and he had had he privilege of being associated with one of the engineering aspectors of the Ministry of Health, Mr. J. R. Taylor, M.A., I. Inst. C.E., and with Mr. Peirson Frank, M. Inst. C.E., the present hief Engineer of the London County Council, in looking into the roblem in a broad, general way. They had issued a Report at the eginning of 1935 which dealt with the possibilities of a unification of ontrol over an area of about 1,928 square miles, roughly corre- ponding to a circle having a radius of 25 miles from Charing Cross.

The West Middlesex scheme dealt with the drainage of an area f about 160 square miles, lying within the 25-mile radius to hich he had just referred. The opinion expressed in the Ministry f Health Report was that in the whole of the circle having that adius there should be ten or fewer centralized disposal-works. He herefore welcomed the West Middlesex scheme as a step in the irection then recommended. How far the proposition that the mitation of a drainage-area should depend upon watershed boun- aries and not upon local government boundaries had been observed the works in question he had not had time to ascertain, but he nought that, in that particular case, nature had been kind, or rather aat the local government areas happened to follow very closely the atural watershed areas. He doubted whether that would be the

Sir George Humphreys.

case if and when centralization took place in further districts, an anyone who looked to the future would have to try to persuad the county councils concerned to merge their interests together.

There were a great many points of interest in the Paper to anyon who was concerned with large drainage-works, and he thought tha the speed with which the work had been done was really remarkable There were also, as would be seen by a close examination of th Paper, many details of methods of execution which were mos ingenious and efficient. In Appendix II it would be seen that ther were about 60,000 yards of sewers of 3 feet diameter or less. H did not know whether the Middlesex County Council was assumin responsibility for the maintenance of all the small sewers, or whethe some arrangement existed akin to that which existed in London where the control and maintenance of the smaller sewers were veste in the borough councils.

He had visited the works, and had been impressed by the use o the concrete segments for tunnel-lining. In Appendix I (the Tabl of costs) it was made clear that the diameter of the cast-iron sewer was the diameter of the inside lining. Was that true also of th brick sewers ?

Mr. Temple.

Mr. F. C. Temple observed that it was stated that the size of th sewers was based on the population to be served. The size of th sewers in the West Middlesex scheme corresponded with a run-o due to a rainfall of 0·10 inch per hour on the whole area. In Indi it was more usual in designing sewers to ignore the population an to work on the area alone, allowing for the total run-off ; in a grea many cases that was taken at 0·25 inch per hour, 2·5 times th run-off allowed for in the West Middlesex scheme.

The spacing of the manholes tended to be much closer in India i the case of small sewers which had to be rodded out, as the rodma had not the physique to rod more than 150 feet. It was state in the Paper that manhole shafts 2 feet 9 inches in diameter wer satisfactory, but that 2 feet 6 inches was inconveniently small. Tha was in agreement with Indian practice, but a large man could get i by himself through a 15-inch cover, and he could get out with hel

It would be of interest to know just how the sewage behaved a the backdrop by-pass cascades. Step cascades and by-pas cascades had been used in India for over 15 years, and from *Fig.* (p. 484) he would have expected that when the branch sewer wa running full the sewage would jump all the steps. It might n matter if it did, because there would probably be enough water i the main sewer to cushion it.

The method of returning screenings after disintegration to t incoming sewage was one of those things which appeared to be a

ovious arrangement once it was done, but it did not seem to have Mr. Temple.
een thought of before. It was very interesting to look back on the evelopment of the channel grit-catcher. It seemed to have been sed first in Pretoria by Jamieson, and then it had been used in ndia. It had been discussed in London in 1924, and it appeared s though it would be adopted as standard practice. The velocity f 1 foot per second which had been adopted at Mogden agreed with ndian practice, and the arrangement for controlling the velocity eemed to him as being likely to be the best yet devised. The ngth of the channels, however, was open to question; they seemed be about 100 feet long, and whilst the first 30 feet was essential nd the second 30 feet might justify its existence, it was doubtful hether the amount of extra grit brought down in the remaining) feet was sufficient to justify the extra cost involved.

He believed that the right methods were now being adopted for it-catching, but he did not agree with the methods of dealing with e storm-water. In India attempts had been made to get down to st principles and it was possible that those would apply in England so; if they did, a great deal of money could be saved on future orks. Storm-water tanks in England were still calculated on the sis of detention-period. That was satisfactory when a storm did ot last long enough to fill them, because then all their contents were umped back into the sewage and dealt with in the disposal-works, ut when a storm lasted long enough to overflow the storm-tanks, ey changed their essential function and became practically grit-tchers; their function was then to abstract from the sewage as uch as was reasonably possible of its constituents which would do rm in the river into which they were discharged. Those con-tuents were organic putrefied solids of which the most serious ere those heavy enough to settle on the bed of the river and to lie ere and cause trouble. Storm-water tanks would not appreciably fect the matter in solution, and they would not retain floating ids which had passed the screens unless they were provided with um-boards, but they would retain such solids as settled during the ssage of the sewage through them. Grit-catchers were formerly lculated on the basis of detention-period, but it had now been cognized that they should be designed to reduce the velocity of a vage to a certain figure for a certain distance of travel, because e solid matter came down in a curve. In the same way, he lieved that what was wanted in storm-water tanks was a certain ocity for a certain length of travel. He had found that 0·1 foot r second was the lowest velocity that was worth while, and that out 120 feet was the maximum length of travel that was worth nsidering. Any additional cross-sectional area which reduced the

Mr. Temple. velocity below that figure, and any additional length, did not produ results commensurate with the additional cost of making larger tank

Applying those figures to Mogden, where the rate of flow w 80,000,000 gallons a day, or roughly 150 cusecs, a total width 150 feet was required with flow 10 feet deep at a velocity of 0·1 fo per second. Whether the tanks at Mogden, which in the aggrega were eight times that width, 2 feet deeper and nearly twice as lon would do enough good by catching small storms to justify the construction was, he thought, doubtful. He would be very su prised if the whole flow, when the tanks were full, could not be p through one single tank and give almost as good an effluent, a certainly an effluent good enough for practical purposes.

Mr. Frank. Mr. T. Peirson Frank remarked that some 6 or 7 years ago t London County Council had been endeavouring to decide wh process they should adopt, and they had put in what they had term an "experimental" plant to treat 10,000,000 gallons of sewa a day; the process, while not agreeing in detail with that adopt in the West Middlesex scheme, did coincide with it to some exte The London scheme had not the secondary sedimentation-channe but the reasons given for those in the Paper thoroughly justified t work which had been carried out; Mr. J. D. Watson had referr to them in his opening remarks.

The London scheme had not proceeded as the West Middles scheme had done, as the London County Council were sometim unfortunate in obtaining grants from the Ministry of Transport, a they had not been so fortunate with the Ministry of Health as h the Middlesex County Council, who were to receive a grant approa ing £3,000,000 if the work were paid for within a period of 15 yea If the London County Council had had that inducement, he w sure that they would have been much further ahead with the wo which were at present in hand. There was, however, another reas why Middlesex should be favoured rather than an authority lo down the river: London was greatly dependent on the river-wa passing that densely-populated neighbourhood, and it was grea to the benefit of London that Middlesex should improve the qual of its effluent.

He could not quite understand the statement on p. 517 t "Return-sludge has been allowed for at rates up to 50 per cent. the dry-weather flow." That seemed an enormous quantity, probably the plant was not working to full capacity at the pres time, but in the last sentence of the same paragraph the Aut said: "It has therefore been arranged to return up to 100 per ce of the dry-weather flow, of which 50 per cent. is sludge and per cent. return-effluent." That looked as though the dilution

ing to be done in dry weather and during such time as the flow Mr. Frank.
the works was much below what would be expected with the
ded population. It would be interesting to know what percentage
sludge that would give after 1 hour's settlement. In the case of
e London plant which had been working for some 6 years, quan-
ies of activated sludge, ranging from 5 per cent. up to 15 per cent.
d been tried, and the plant had been working for some considerable
ne with 10 per cent. of sludge.

The rate was given on p. 555 as 6·0*d*. in the pound. The total cost
s stated to be about £5,156,000 and from p. 465 the Unemployment
ant Committee's contribution would appear to be about £3,200,000
at was, ". . . based on 75 per cent. of the loan charges on
proved expenditure for 15 years, £4,290,000 being the maximum
m ranking for computation of the grant . . ."). The figure of
,200,000 would depend upon the period (not stated) within which
loan was repaid. It looked, therefore, as though Middlesex
uld have to provide a sum approaching £2,000,000. If that sum
quired the rate of 6*d*. in the pound, then it would seem that if no
nt had been forthcoming works costing £5,156,000 would have
quired a total charge of about 1*s*. 4*d*. in the pound—that was,
uming that the rate of 6*d*. was to meet debt-charges only, as
Paper appeared to indicate.

London was also faced with the problem of sludge-treatment.
out 2,000,000 tons of sewage had to be disposed of per annum,
d, probably because of the quantity, it had been possible to
pose of it at an operating cost of 5·7*d*. per ton for the last 12
nths. Capital charges or debt charges took 1·7*d*., so that the
al cost was 7·4*d*. That involved taking the material about
miles out to sea, and there had been no complaints for a number
years. That had to be compared with the cost of sludge-digestion.
experimental plant was being put down, but again, lacking the
ucement which Middlesex had received, it had not yet been
npleted.

It was hoped to make use of the energy derived from the digestion
the sludge, and as a result of previous smaller experiments,
was proposed to operate at a higher temperature than that
ntioned in the Paper. The Author gave the optimum temperature
the digestion process as 80° F., but in London, although the
ditions were presumably the same as in adjoining counties, it had
n found that from 85° to 90° F. gave rather better results. What
the cost of the treatment per million gallons in the West
ddlesex scheme? He appreciated that it was bound to be high
first, because provision had quite rightly been made for a very
ch larger flow. Was the figure of 2 cubic feet of free air per

Mr. Frank. gallon (given on p. 516), which it was thought would be reduced 1·4 cubic foot per gallon, likely to be materially reduced in future

It was mentioned on p. 501 that for the extra work referred to t contractors were given a 10-per-cent. bonus on the rates tender for the main contract. That seemed a little generous. He appr ciated that that inducement meant that the work had to be speed up, but he believed that for that type of work the value—cost syste of contract might have been considered with much favour and mig have operated economically. It had the advantage that work cou start even before detailed drawings were completed, as a schedule w given, and on that comparative competitive tenders could be p before a Committee. That would have avoided the trouble referr to at the end of the second paragraph on p. 499. With regard to t construction of backdrops, most of the sections of the work appear to be so designed that some considerable fouling was likely to ari The Author had probably thought of introducing chlorine to avo ill-effects. That had not been done in connexion with backdrops, b in the case of long lengths of sewer in London, even in some passi close to the Institution.

It had been found of great advantage in the disposal works to u chlorinated ferrous sulphate—a mixture rather than a chemi compound.

Mr. Morris. Mr. ARNOLD MORRIS suggested that the West Middlesex drain achievement was of interest because of certain economic or soc aspects, as well as on account of the engineering features of the wo

From the point of view of town-planning, the expenditure £5,250,000 had brought about significant changes in west Middles Twenty-seven sewage-disposal works had been abandoned, and t meant that their sites were freed for other uses; in addition, environs of those sites were made available for uses which w not likely in the vicinity of a sewage-disposal works. One W Middlesex authority had been able to allocate land adjoining one the works for open space and recreation.

He believed that the completion of the trunk-sewers described the Author had affected other land than that near former sewa disposal works. The approval of plans for building developm was bound to have become less of a problem for local authorit who would feel secure in the possession of facilities for the discha of sewage into trunk-sewers of liberal dimensions, and who wo no longer have to fear the results at the usually overworked dispo works. Development had therefore been made easier, with possible improvement in land-values. He would welcome from Author an expression of his opinion as to whether the West Middle system had or had not improved development-prospects in any

the area served. The work in Middlesex had been financed Mr. Morris. tirely out of the public purse, either local or national, and he ggested that in similar future projects careful consideration should given to the possibility of contributions from private interests.

He suggested that main-drainage schemes would have equally neficial results on development-prospects in other parts of greater ndon, in south-east Lancashire and in the West Riding of York-ire, as well as other great benefits from joint drainage. It was obable that in west Middlesex the granting of Government sistance had turned the scale in favour of adopting the scheme, d since similar grants were not so likely in the immediate future, nemes for other areas could well include provisions for contributions om other sources, assuming that benefit could be proved.

Most local authorities now possessed the power to delay building-velopment where the sewerage-expenditure involved would be emature. On the other hand, the landowner who had land pable of development only after the construction of a main wer was generally willing to acknowledge, in hard cash, the value him when public money was spent on sewerage. In a recent ample in Surrey an owner had desired to develop land, the sewerage which would have led to an expenditure of £13,000 by the Council, her land also being served by the system. After having paid £270 r acre for the land the owner had offered a further £60 per acre to e Council if they would make the sewer, making the land worth 30 per acre to him. It was by no means extravagant to say that nen houses were built on that land, the price paid by freehold rchasers would be £700 per acre and upwards, for sites alone. ssuming, however, that that Council made that £13,000 sewer, her landowners were likely to secure the benefit without contribu-on, and it appeared most desirable that power to make charges on vners for sewerage should be made available. A scheme as large that of West Middlesex would be the subject of a Private Act, d charging powers could be included. Similar powers had already en given by Parliament, and he contended that the principle had en tested sufficiently to justify a public general Act.

He suggested that any engineer responsible for expenditure on iblic works which might increase the value of private property ould in future examine very closely the prospect of recovery of yments from those who benefited.

Mr. R. G. Hetherington said that twenty-seven existing schemes Mr. Hetherington. d been brought together into one, and that it was in that direction at progress in regard to drainage questions which arose in thickly-ppulated areas was to be looked for in the future. The West iddlesex scheme had ante-dated the important Report on Greater

Mr. Hetherington.

London Drainage which had been prepared by Sir George Humphrey, Mr. Peirson Frank, and Mr. J. R. Taylor of the Ministry of Healt but although it was prior to that Report it did in fact implemen as far as its own area was concerned, the recommendation of th Report to concentrate the sewage in a limited number of big work There was at the present time a Bill before Parliament for the co centration of the sewage-works in the valley of the Colne, whi adjoined the area of the West Middlesex scheme; and there we others, proceeding chiefly by Provisional Order procedure. Tho schemes were all more or less in line with the Greater Lond Drainage Report, although they did not go so far as that Repo suggested, and it seemed to him that it was going to be a matter proceeding by steps and not of doing everything at once. The We Middlesex scheme was the first drainage-scheme undertaken by county council for concentration of existing sewage-works, excludi the London County Council, which was an entirely special case. was interesting to note that the Colne Valley scheme was also bei undertaken by another county council.

The advantages of concentration were many, including t abolition of separate and individual sewage-works. A sewag works, however well it was run and however free from nuisance was, was not looked upon exactly as an asset to the district whi immediately surrounded it, and it did not add to the value of th area; it was also apt to restrict the use of the district for develo ment purposes. Another great advantage was that, by conce trating the sewage from a large number of districts, the quality the sewage which came down was steadied, whilst the sudden d charge of trade-waste, which might entirely upset the working o moderate-sized works, could be dealt with by works of a sufficien large size. The West Middlesex scheme accepted from each in vidual authority six times the dry-weather flow. What did th mean in the multiplication of the dry-weather flow which actua arrived at the works? The area covered being so large, the effect rain-storms might be very much spread, and it might be found to the case that, by those large concentrations, a certain amount at a rate of the very big variation in the rate of flow which had at prese to be faced in connexion with sewage-works would be eliminated. further advantage arose from the ability of a big works to carry really efficient staff. A works on the scale of that under discuss could afford to carry a staff not only of engineers, but of chemists a even bacteriologists whilst in service, which was outside the scope the small works and authorities.

One disadvantage of concentration was that of cost. In the ea days of such schemes the cost was apt to be heavy in comparison w

e cost of individual works. That disadvantage was probably llified after a few years, and the cost of the works then was less an would have been the cost of the individual works at that ıge, but it was not always easy to convince authorities to spend . to-day in order to save 3*d*. to-morrow. He thought that there ıs a good deal of useful work to be done in trying to convince the blic of the financial advantages which were ultimately gained by ch works, even if those advantages were not visible in the first 2 3 years. The cost, which was an important factor in big schemes, s such that it would probably exclude a scheme which contained ıumber of "sterile" sewers; by "sterile" he meant sewers which ved and passed through uninhabited areas. Under such conions it could not be hoped, at any rate at present, to obtain large ıcentrations of sewage from different areas. Mr. Hetherington.

The West Middlesex scheme had been fortunate in the sense that had been started at a time when Unemployment Grants were ailable, and it had obtained a very substantial Grant from the vernment. He noticed that in the course of the Paper there re one or two references to the speed which had had to be employed carrying out the works. That had been inherent to the Unemploynt Grants, and those Grants had been given through the Ministry Labour with the intention of producing the maximum amount of rk in the minimum amount of time; they had been given in urn for the work being undertaken in perhaps not the most comtable or the most easy of circumstances. To a certain extent ıt fact might be put against the Grant.

The West Middlesex works possessed a full equipment of recording ɔaratus, and he hoped that at a later date, when the works had en running for a few years and after adequate use had been made the recording apparatus, a further Paper would be written, stating at results had been obtained; a good deal of information might n be given, which would be of great value in assessing the value future schemes for concentration of sewage-works.

Mr. Townend. Mr. C. B. Townend remarked that, as stated in the Paper, the ıstruction of the works had been timed to be completed on the 1st tober, 1935. In the case of the main sewerage, with the exception one length of about 50 yards delayed by exceptionally wet ground, whole of the system had in fact been completed on that day. Mogden, in spite of a 70-per-cent. increase in the work originally ıtemplated, construction had been sufficiently advanced to make first diversion of the local sewage from the Heston and Isleworth trict on the 14th December, 1935, or only about 10 weeks after appointed day. Within 1 week sufficient bacterial energy l been built up to produce an effluent complying with the pre-

Mr. Townend. scribed standard of purity, and since that time the performance o the plant had never deteriorated. After a trial run of about 6 week duration it had been decided to embark on the main programme o sewage-diversion from the twenty-seven old works and, by the en of May, the sewage from over 1,000,000 people had been connecte to the new system. The organization of that change-over, unprec dented in magnitude and involving about 100 connexions of variou sizes throughout the drainage-district, had been contemplated wit a certain amount of apprehension, but the operations had been carrie out with remarkable smoothness and absence of serious difficult By accumulating a reserve of activated sludge in the reconditionin units, always available to take charge of the increasing sewage-flow the quality of the effluent had been maintained continuously at high level, even although on occasion the sewage of an addition 100,000 people had been connected overnight.

The results for the first 7 months of full operation from t 1st June to the 31st December, 1936 (214 days), were set out in t following Tables :

Population served	1,070,000
Dry-weather flow (estimated)	43,000,000 gallons per day

SEWAGE-FLOWS.

	Total flow: gallons.	Average daily volume: gallons.	Percentag
Sewage received at works . .	10,842,500,000	50,660,000	100
Volume treated fully in purification-plant	10,819,000,000	50,550,000	99·8
Volume treated partially in storm-water tanks . . .	23,500,000	110,000	0·2

AVERAGE ANALYTICAL RETURNS EXPRESSED IN PARTS PER 100,000.

	Sewage.	Effluent from sedimentation tanks.	Effluent from activated-sludge plant.	Final effluen discharged in river Thame (including a storm-wate
4 hours' oxygen-absorption at 26·7° C.	8·51	4·73	0·98	1·00
Ammoniacal nitrogen .	4·06	4·11	0·59	0·60
Albuminoid nitrogen . .	0·92	0·47	0·077	0·081
Nitrous nitrogen . . .	—	—	0·31	0·30
Nitric nitrogen . . .	—	—	2·14	2·12
Chloride as chlorine . .	11·3	11·5	11·0	11·0
B.O.D. in 5 days at 18·3° C.	35·0	19·5	0·67	0·83
Solids in suspension . .	29·2	8·5	0·81	0·96
Putrescibility	—	—	0/206	0/208

Mr. Townend.

PERCENTAGE PURIFICATION.

	Measured by the following tests:—		
	4 hours' oxygen absorption.	Albuminoid. nitrogen.	B.O.D.
Purification, calculated on influent in each case, effected by:—			
Sedimentation-tank processes (a)	48	44	45
Activated-sludge process (b) . .	79	87	98
Combined processes (a) and (b) .	89	93	99
Purification of raw sewage, as determined from analyses of sewage and final effluent discharged into the river Thames (including all storm-water)	89	92	98

The chief points indicated were as follows:—The average dry-weather flow realized very closely approximated to the 40 gallons per head forecast in all preliminary estimates. In the 7 months concerned, the total flow received had been nearly 11,000,000,000 gallons, giving a daily average of rather more than 50,000,000 gallons, or about 18 per cent. in excess of the dry-weather flow. The analytical results, for which he had to thank the chief chemist, Mr. W. T. Lockett, indicated a medium-to-strong sewage, predominantly domestic in character, having a biochemical oxygen demand of about 35 parts, and containing about 30 parts of suspended solids per 100,000. Although the effluent was discharged into the tidal section of the river, no advantage had been gained by that fact, and the standard of purity laid down by the Act was the same as would be required in the case of a small inland fishing stream.

Throughout the period a well-nitrated effluent of exceptionally high quality had been produced, the purification effected being about 90 per cent. on the 4-hours' oxygen-absorption and albuminoid-nitrogen tests, and about 98 per cent. on the biochemical-oxygen-demand test. The suspended matter had consistently been less than 1 part per 100,000, whilst the normal average figure for the biochemical oxygen demand was about 0·5 part per 100,000, or only one-quarter of the amount allowed. The nitrogen as nitrate had averaged over 2 parts per 100,000, which indicated that some ,000 tons of oxygen had been discharged into the river during the months, the whole of which had been available to take charge of the heavy pollution occurring further downstream.

The air-consumption of the aeration-tanks had averaged 1·5 cubic foot per gallon of dry-weather flow, requiring approximately 38 HP. per million gallons on a basis of dry-weather flow. Those figures

Mr. Townend. would answer Mr. Peirson Frank's question. In giving them, it should be noted that the whole policy of works-operation up to the present time had been to produce a superior effluent. There was no doubt that, although the power-consumption was very reasonable, the margin of quality of effluent was such that the power-consumption could be reduced if necessary. On the other hand, at the Mogden works, with ample power available from internal sources, there was no need at present to modify operations.

A factor of outstanding importance which had recently attracted a great deal of public attention was the question of storm-water. The average flow of 50,000,000 gallons per day indicated that a volume of about 1,600,000,000 gallons of storm-water had been received into the system during the 7 months, but so great had been the equalizing effect of the large sewage-district that the storm-water tanks had been brought into action on 14 days only out of 214. On the majority of those occasions the whole of the impounded flow had been subsequently returned for full treatment in the purification-plant, and on only 5 days was any effluent from the storm-water tanks passed direct to the river. The total volume thus discharged in the 7 months had been 23,500,000 gallons—a phenomenally low figure, representing only 0·2 per cent. of the total flow. The maximum flow during that period had reached a rate of 190,000,000 gallons per day, or nearly 4½ times dry-weather flow. He thought that that answered Mr. Hetherington's question. During the period of heavy rain in the first 2 months of the present year, however, the relief siphons on the river Brent had come into action on several occasions (thus creating an artificial flow), and the maximum flow on 2 days had reached a rate of 280,000,000 gallons per day.

With regard to sludge-disposal, the total quantity produced, including both crude and surplus activated sludges, had averaged about 2,000 tons per day, having a water-content of about 95 per cent. The whole of that had been dealt with successfully in the digestion-plant, which had been brought into use without undue difficulty. The plant had been continuously maintained at a temperature of from 80° to 85° F. by utilizing the waste heat from the power-house, which had been much more than sufficient for the purpose.

In the power-house the engines had been converted one by one from oil-operation as methane gas from the digesting sludge had become available. During the 7-month period the actual consumption of methane for power-purposes had been 136,000,000 cubic feet. At the present time, apart from mechanical adjustments which were not yet quite completed, the power-load of the entire works was

being produced from gas-fuel, the daily consumption on many occasions having exceeded 1,000,000 cubic feet. The highest figure (1,150,000 cubic feet) had been reached on the 28th February, 1937, whereas the most optimistic forecast, as mentioned in the Paper, had been 625,000 cubic feet. The present power-output was equivalent to 15,000,000 electrical units per annum. Mr. Townend.

About 9 years ago he had been privileged to be the joint Author of a Paper on the first stage of the sludge-gas power-plant at Birmingham.[1] It had been little thought at that time that such great developments would take place in such a comparatively short time. If a value of only 2*d*. per therm were placed on the Mogden gas, the annual value at the present rate of output would be sufficient to meet the entire cost of sludge-disposal both for capital and operation. In answer to Mr. Frank's question, it might then be said that the cost of sludge-disposal per ton was nil.

A Paper had recently been given before The Institution on broadcasting transmission.[2] It was interesting to reflect that the power produced at Mogden would supply practically the whole of the B.B.C.'s annual requirements throughout Great Britain. It had been stated that, of that power, all that a listener in America received was enough energy to raise a fly 7 inches in 1 year. By way of contrast, if the whole of the West Middlesex population of 1,000,000 people were placed on a gigantic lift at Mogden, it would be possible to raise that lift by the power produced in the works to a height of 1,000 feet in a day, or 70 miles in a year!

He felt it a great privilege to be connected with the works, and to have been a member of the team who had carried out their construction, but no team, however efficient, could work effectively without the inspiration of a chief such as Mr. John D. Watson, who had been so ably partnered by his son. It could not be too strongly emphasized that schemes such as West Middlesex Main Drainage had only been rendered possible by the work of the great pioneers of the past, and of those there was no finer example than Mr. Watson, who had for many years been regarded as the doyen of his branch of the profession.

Mr. H. C. Whitehead said that a little more than 60 years ago Mr. Joseph Chamberlain, the then Mayor, had decided that parochial management of sewage-disposal around Birmingham had to give way to regional control, and he had persuaded nine neighbouring authorities to collaborate with Birmingham in forming a Joint Mr. Whitehead.

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[1] F. C. Vokes and C. B. Townend, "Power-Gas from Sewage-Sludge at the works of the Birmingham Tame and Rea District Drainage Board." Minutes of Proceedings Inst. C.E., vol. 226 (1927–28, Part 2), p. 4.

[2] Sir Noel Ashbridge, "Modern Developments in Broadcasting Transmission and Television." Journal Inst. C.E., vol. 5 (1936–37), p. 349. (March, 1937.)

Mr. Whitehead. Board. Mr. Whitehead was sure that the Author of the present Paper would be amongst the first to agree that the wonderful works that he had described might be regarded as one of the consequences of Mr. Joseph Chamberlain's far-sighted action.

It was less than 16 years since he had designed and constructed for the Author's father an experimental plant at Birmingham to obtain power by the gas evolved from sewage-sludge. Practically no data had been available, and although the experiment had been successful in indicating that power could be obtained from sludge-gas, it had not been until 1927 that he had been able to devise a means of collecting and utilizing the gas in a safe and economical manner. In that instance a 150-B.HP. engine had been used, and it had generated about 500,000 units of electrical energy per annum. Plants had since been erected in many places, utilizing a previously untouched source of power which, although small as compared with coal and oil, had the merit of being inexhaustible. The Mogden power-installation was at present the largest of its kind in the world.

Owing to site-restrictions, sludge-digestion was carried out partly at Mogden and partly at Perry Oaks, and that followed the practice in the main works at Birmingham. The method had, however, a disadvantage in preventing the re-use of the alkaline liquor, which was strongest in the later stages of digestion, and which, if returned to the primary tanks, was of use in raising the alkalinity there, and thus accelerating the speed of digestion. It might be that in any future extension of the West Middlesex works the Author would—whilst not increasing the total digestion-capacity of 4 cubic feet per person—deem it worth while to increase the proportion used for primary digestion. That would undoubtedly result in a higher average alkalinity in that part of the plant, and would give more stable working conditions.

It was rightly held that the activated-sludge process was more sensitive than the percolating bacteria-bed, but from his own experience he would suggest that the disturbing effects of a strong industrial sewage might often be counteracted by greater dilution with effluent than was now usual, and he was pleased to see that provision for dilution of the influent had been made at Mogden. Whether the activated-sludge process was or was not applicable to any given sewage depended not so much on the magnitude of the ultimate biological demand of the liquor to be treated, as on the avidity with which that liquor demanded oxygen. In some instances the very rapid initial avidity for oxygen of the sewage-liquor could not be satisfied quickly enough by ordinary means of aeration, and under those conditions the process required modification in the manner he suggested. Greater dilution might be effected either b

Mr. Whitehead.

increasing the volume of water carrying the returned activated sludge, or, preferably (where sedimentation of the sewage was carried out in two stages), by adding settled effluent at the inlet to the second stage. By the latter means any oxidized nitrogen in the effluent was utilized to the greatest advantage. The possibility of dilution had been first indicated by the Royal Commission when they had pointed out that two volumes of weak sewage could be more easily purified on a percolating filter than one volume of sewage double the strength of the weak sewage. In suggesting a greater use of dilution as a preliminary to oxidation processes, he had in mind not only experience in Birmingham but the recent work by the Water Pollution Research Board in the successful treatment of milk-waste liquors. Many trade-wastes and some sewages might with advantage be given dilution with their effluent to the extent of 200 or 300 per cent. as a preliminary to biological purification.

Earlier in the discussion Mr. Frank had compared the provision for returned activated sludge of up to 50 per cent. at Mogden, with that of 10 per cent. used by the London County Council. For complete treatment by activated sludge a return of up to 50 per cent. was quite common, and in certain circumstances it might be increased. For partial purification by activated sludge, such as was employed in Birmingham and by the London County Council, a much smaller proportion of sludge was sufficient, and returns of from 10 to 15 per cent. were usual.

He disagreed with Mr. Temple, who had suggested that storm-water tanks ought to be designed on a flow-velocity basis. A storm-water tank could protect the river in two ways, (*a*) by impounding moderate discharges of storm-water until they could be purified in the oxidation plant (such moderate discharges were often quite as strong as, or even stronger than, the sewage itself), and (*b*) by affording means of sedimentation to relatively large rates of flow of storm-water. Only tanks of large storage-capacity could perform that dual function. It was safe to assess the value of a storm-water tank of 12 hours dry-weather-flow capacity at three or four times that of a tank of 6 hours' dry-weather-flow capacity. The magnitude of the West Middlesex drainage area, and the immense volume of sewage to be treated, were valuable safeguards to the purification-processes in operation at Mogden, in that they tended to smooth out fluctuations in volume and strength of the sewage to be treated, and there was added safety in the great variety of trade-wastes usually found in a large drainage-area. In his opinion the Mogden works would be able to work satisfactorily with a considerable increase in sewage-strength, and there was no doubt that their successful

Mr. Whitehead. operation during the next few years would do much to encourage a much-needed development of regional sewerage and sewage-purification.

Mr. Morgan. Mr. W. H. Morgan observed that the highest compliment that he could pay to the Author was to state his opinion that, despite the speed with which the great work under discussion had been carried out, no modification of any importance appeared to be desirable.

The first connexion had been made on the 14th December, 1935, and by May, 1936, all authorities within the drainage-area had been connected up. There had been slight troubles in the early stages, because everything had been new and some parts incomplete, but to-day the work was going very smoothly and most successfully. The results obtained were beyond the most optimistic expectations. The effluent was of a very much higher standard than that demanded under the Act, and the amount of gas produced was now over 1,000,000 cubic feet daily.

With regard to cost, 1937–1938 would be the first full year for budget-purposes, and the estimated cost to be met by the rates was 6·04*d*. in the pound. The County Council at their last meeting had decided to levy a rate-charge of 6*d*. in the pound. The total gross estimated rate-cost, ignoring the Government Grant, was 10·03*d*. in the pound, made up as follows:—main sewerage 4·21*d*., purification-works 4·57*d*., indemnification of local authorities (including recoupment of their loan-charges, compensation for displaced officers, etc.) 1·25*d*. The Unemployment Grant amounted to 3·99*d*., giving the net total of 6·04*d*. in the pound. He wanted to emphasize the fact that the cost of the purification-works themselves amounted to 4·57*d*. in the pound; that was the complete charge, without any Government assistance, and, in the judgement of those concerned, it would be a peak figure and would undoubtedly fall. A penny rate brought in £43,400 in the drainage-area.

The administrative staff numbered 31, and 185 were on th weekly-wage list. Administrative staff included all persons paid o the monthly-salary list. The number of men employed on runnin the machinery, pump-houses and compressor-house was 40; ther were four sewerage-gangs of 6 men each and 1 foreman, and th permanent gang at Perry Oaks numbered 20. The men directl employed on the purification-plant numbered 70, leaving 30 men o various odd jobs. At the Perry Oaks sludge-disposal works it wa estimated that some 70,000 cubic yards of sludge would be shifte and tipped next season, and the method of payment which would b used would be the bonus principle. Mr. Whitehead had very kindl allowed one of the staff to spend some days at Birmingham watchin

:he process, and as a result of his report the County Council had lecided to adopt a similar policy. Mr. Morgan.

Apart from the abolition of twenty-seven sewage-works, other ıdvantages in the drainage-area had accrued as the result of the cheme. The County Council of Middlesex was a Rivers Authority, ınd as such was responsible for the proper maintenance of its nternal rivers and streams. For some years, owing to the phenonenal growth of the population of Middlesex (which continued ıt the rate of 1,000 persons per week) they had been seriously conerned with pollution; in fact, prior to the inauguration of the vorks, certain rivers had been no better than open sewers. That ıad not altogether been the fault of the local authorities concerned, ecause they had not been allowed to spend any money to improve heir systems, as the Ministry of Health had known that the present cheme was being prepared. That condition of affairs had now lisappeared, and the rivers were what they should be; if any ollution did occur from isolated sources it was readily detected and apidly corrected. By the scheme, about twenty works had had heir wastes led into the sewers.

Despite the abnormally wet winter the county had not been roubled by floods. That was due to a very large extent to the ıain-drainage scheme. Once or twice recently the flow at Mogden ıad exceeded 280,000,000 gallons per day. Quite a large proportion f that effluent would have been taken into the streams, and during he progress of the work Messrs. J. D. and D. M. Watson had very isely advised the Council to put in siphon spillways at special points. 'hose had done excellent work during the very wet weather, and ad relieved the river to the extent of about 1,000,000 gallons per ay. Therefore, as well as having their sewage treated in the most p-to-date method possible, the County Council and the people living ı the area had been relieved of flooding.

Dr. H. T. Calvert remarked that in one respect the task of esigning and constructing the Mogden sewage-disposal works had een easy: the designers and contractors had had a virgin site and ad not had to make their designs fit into some already existing orks. The works had had to be designed for a very much larger opulation than they were now serving, and the difficulties resulting om slow-flowing septic sewage were well known. The question of entilation of sewers might have received more consideration, for e could not help thinking that knowledge on that subject was not ll that it ought to be, especially having regard to the experience ı connexion with the Mersey tunnel. The purification-works mbodied several design-features which, if not entirely orthodox in eneral practice, might be worthy of becoming so. Grit-chambers Dr. Calvert.

Dr. Calvert. to produce clean grit, primary and secondary sedimentation-tanks sludge-digestion in two stages, and the continuous use of storm water tanks, were all features which deserved comment. On p. 51 the Author gave the prompt removal of sludge as a reason fo adopting two-stage sedimentation, and he stated that "For thi reason, amongst others, two-stage sedimentation was provided." He would be glad if the Author would enumerate some, at any rate of the other reasons. Another feature of the works which deserve commendation was that to which Mr. Hetherington had referred namely, that the works were amply provided with meters an recorders. Works of that description could not be too ampl provided with those aids to new knowledge, and he would like t emphasize how many sewage-disposal works had not come up t expectation because the basic data on which they ought to hav been designed had been totally inadequate for the purpose.

At the official inauguration of the works in October, 1936, th Minister of Health had expressed the hope that the works woul not be the last to deal with sewage-disposal on a regional basis, an Dr. Calvert was sure that all would agree with him.

He hoped that the present Paper might be followed in 3 or 4 year by another Paper read before The Institution and describing th results of the operation of the works. It had been his privilege o more than one occasion to visit the works, and he had added to hi knowledge of the problems of sewage-disposal on every such occasion

Mr. Cox. Mr. S. W. Cox remarked that his firm were very proud to hav had such a large share in the carrying out of the West Middlese scheme. It was very gratifying that the success of the system of reinforced-concrete tunnelling on the first two sections of sewer to be constructed had been such as to justify its subsequent us on many miles of sewers. The Author, on p. 489, gave the rate of progress for different diameters and methods of construction The progress in 24 hours for the concrete segments was, howeve based on working two 10-hour shifts, whereas the progress fo driving one of the 12-foot diameter cast-iron tunnels (the on which his firm had constructed) was based on using three 8-hou shifts. The reason for using the different times of shifts had bee that each had been found the most satisfactory way of employin the available labour; it had been possible to use several workin faces at once on the long lengths of the concrete tunnel, wherea in driving the cast-iron tunnel only one face had been availabl and therefore the men had had to be concentrated on that, workin three 8-hour shifts.

The methods of construction which his firm had employed o the disposal-works on contracts "M.3," "M.4," and "M.5" we

eferred to on pp. 546 and 547. A member of his firm had visited Mr. Cox
he United States just before they had commenced work on contract
M.3"; whilst there he had visited large sewage-works then under
onstruction, and they had therefore had the benefit of the knowledge
hich he had obtained, but that had not led them to modify to any
xtent the lay-out of plant and method of construction originally
ontemplated. The possible advantages of the use of cable-ways or
elt-conveyors in the placing of the concrete had been very care-
ılly considered, but it had been decided that derricks were of more
eneral use, as well as being more flexible than belt-conveyors or
ıble-ways, in that they could handle the concrete, the excavated
aterial, the shutters, and the heavy pipes.

In conclusion, he desired to say how much his firm had appre-
ated co-operating with the Engineers' staff, both in the office and
ı the works, and in the carrying out of a scheme which had been
very difficult.

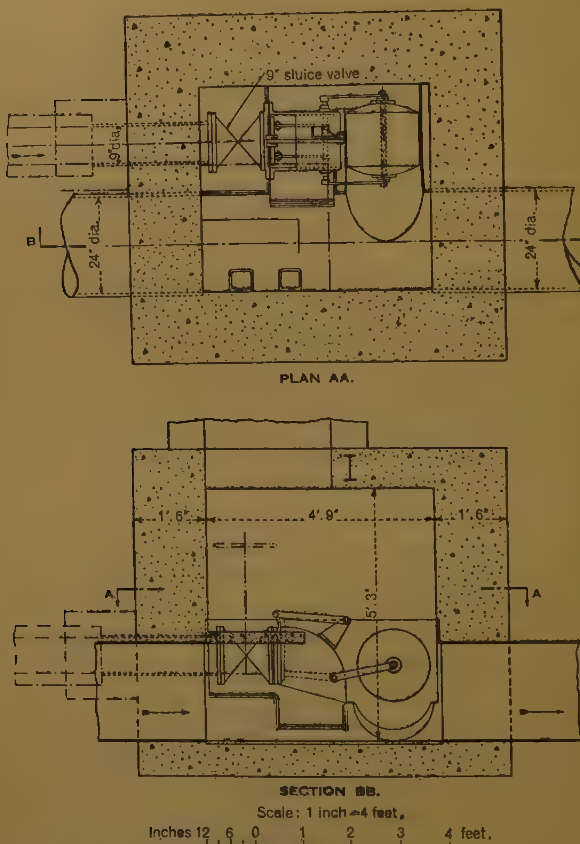
Mr. H. D. MANNING said that on pp. 555 and 556 it was stated that Mr. Manning.
l. in the pound was the rate-charge in substitution for the old
tes for sewage-disposal and purification, and that "... large
vings may be expected both on construction of works and by
ason of cheaper development of local sewerage." He thought
at the effect on local sewerage was worth greater emphasis. In
actically every case the trunk sewers followed lines where either
ere had been an existing sewer or where a future sewer would have
en required. That followed from the fact that they ran in the
lley-lines. In the former case the existing sewers had been over-
xed and would have had to be renewed, and in the latter case the
w sewers would not now be required. In at least one case the
ed for the complete reconstruction of a local intercepting sewerage-
stem had been eliminated. He estimated that the savings
nounted to some hundreds of thousands of pounds.

The principle of betterment already applied in the case of private
reet works sewers, and he believed that certain private Acts had
en passed lately enabling local authorities to apply the principle
all new sewerage construction, but in the case of the bigger
hemes there were certain special difficulties. The fact that a
ınk sewer was laid down a valley-line did not help the landowner
mediately because he had not any right of discharge into it; he
d to work through the agency of the local authority. That was
e case in West Middlesex, at any rate. For administrative reasons
ivate connexions could not be allowed all the way along, and
erefore the only connexions were those with local authorities'
vers. Thus the landowner was only helped indirectly and had to
it for the local authority to lay the necessary sewer. It appeared

Mr. Manning. to him that if the principle of betterment were to be applied to suc schemes it would have to be through the agency of local authoriti rather than by direct contribution from the landowner to the trun sewerage authority.

The special problem which had arisen out of the depth and si

Figs. 43.

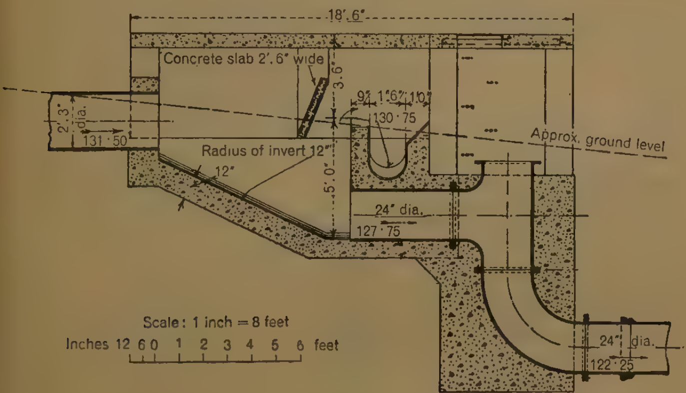


STORM-WATER INTAKE.

of the sewers was a rather unusual feature of the scheme. Temple had referred to the by-pass cascade (*Fig. 8*, p. 484), and suggested that the flow might jump the cascade when the bra was full. Mr. Manning would like to point out that the size of branch in that particular connexion had been dictated by c structional reasons. It had been built in tunnel and it had not b

ssible to make it any smaller; actually, it would never run full, Mr. Manning.
that Mr. Temple's query was answered in that way. The spiral scade shown in *Fig. 11* (p. 487) had been designed for storm-ater only. It might have to carry up to 250 cusecs during a orm but practically nothing in dry weather, when any small flow ould go down the drop-pipe and would leave the steps clear for cess and inspection. The same applied to the step cascade 'ig. 12, p. 487), which was for surface-water only. That fact ade a considerable difference when considering previous speakers' marks about the fouling of those back-drops. The system had t been long enough in operation to form a final opinion, but he

Fig. 44.

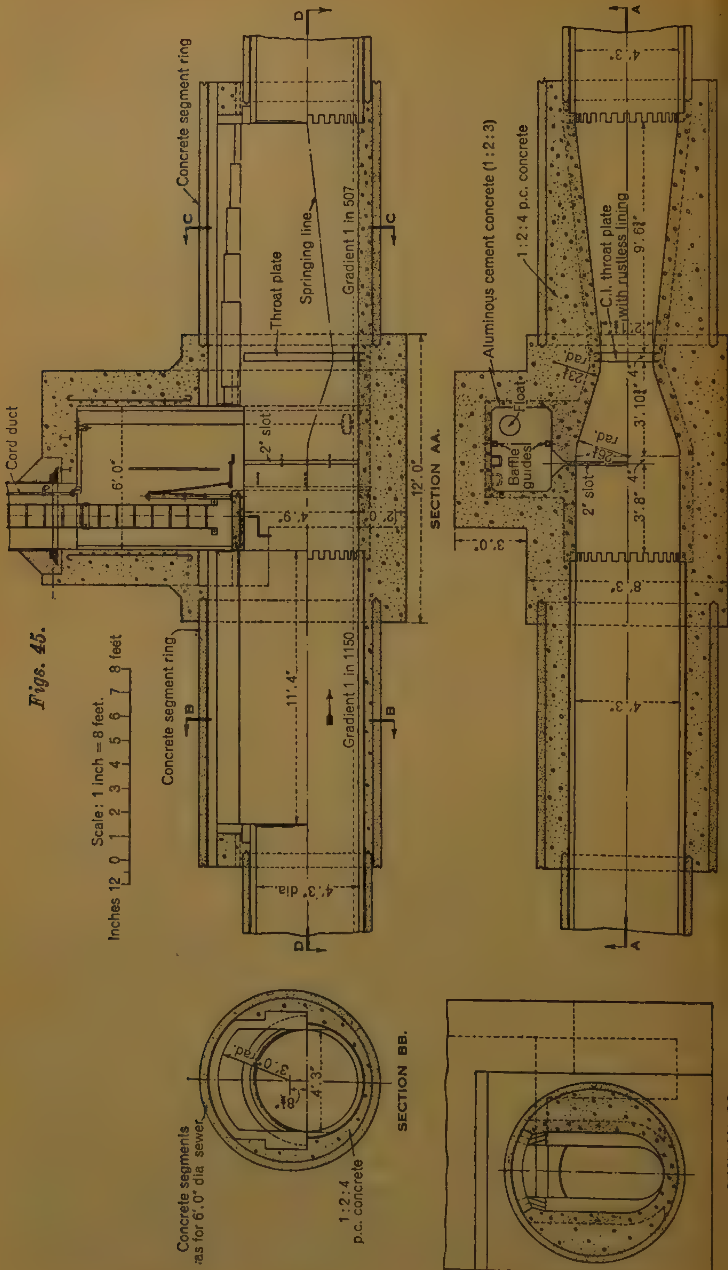


STORM-WATER OVERFLOW WITH ORIFICE CONTROL.

1 seen many of those drops in operation and they appeared to operating in the manner expected.

With regard to the design of the drops, the questions of noise 1 velocity were important. At one drop in particular, there was ioise like an aeroplane passing overhead; the possibility of noise 1 to be very carefully watched, and it was very difficult to allow it in design. High velocity sometimes occurred at the bottom the drop, causing the sewage from the branch to shoot across the ver at the point of entry. At one of the entries the velocity made lifficult for the maintenance-men to pass up and down the sewer. ose points were of interest in the design of connexions, and they e not mentioned in the Paper. Practically every connexion s a special case, but certain general methods of treatment had been pted as more or less standard; *Figs. 43, 44* and *45* (p. 592) showed ails of typical storm-water intake, overflow, and gauging flume.

Mr. Manning.



Figs. 45.

Mr. C. D. C. Braine remarked that the contracts requiring the Mr. Braine.
ast design had been let first and the more difficult contracts had
een left until later, but that the Paper gave little idea of the con-
uous fight against time. In spite of that fight, however, various
signs had been evolved, including the ring-type drive for the final
parating-tank scrapers, devised in the first instance by Mr. P. G.
nales, the arrangement of the wet well at the pumping station,
d, perhaps best of all and due entirely to Mr. Townend, the solu-
on of the constant-velocity problem in grit-chambers, which was
general application. To Mr. Townend was also due almost the
tire credit for the excellence of the details which was such a
ture of the works at Mogden.

As far as the sewers were concerned, the most striking feature
s the balancing effect on peak-flows of the large sewers. That
s a common but incalculable factor in any large sewerage-scheme,
d the effect at Mogden had been that, despite an unusually wet
ar, only on five occasions between July and December, 1936, had
y storm-water been discharged from the works. That was to
, except on those five occasions all the storm-water discharged
o the main sewers had received full treatment, and that despite
fact that gauging-records showed that some of the constituent
thorities had discharged into the sewers at more than their ulti-
te quota rates. He had not the slightest doubt that that would
quently occur without causing the slightest embarrassment at
gden. Had the new sewers not existed, most of the twenty-seven
all disposal-works would, under similar circumstances, have
charged to the rivers large volumes of only partially purified
rm-water, and, during extra heavy storms, perhaps even un-
ated water. Actually, in West Middlesex almost the whole of
t polluted water was being fully treated, which was clearly of
greatest benefit to those interested in the purity of rivers. It
astounding, therefore, to read in the technical Press that a
ior officer of a big water company which was vitally interested
the purity of local streams had criticized the West Middlesex
eme on the grounds that it, and similar schemes, tended rather
increase pollution by storm-water. Even at first glance that
cely seemed reasonable, and it had apparently been written
er a complete misapprehension.

r. Temple had wondered whether large storm-water tanks were
ifiable. The results quoted seem to prove their complete justifi-
on, but it had to be remembered that Mr. J. D. Watson had been
ocating large storm-water tanks holding 12 hours' dry-weather
for many years past; that was now more or less accepted as
l practice.

Mr. Braine. The syphon shown in *Fig. 6* (p. 479) primed at a level of 49·50 that was, when the screen was just submerged. At that level flood ing commenced downstream. Thus, although the water-compan critic said that the river now was not flushed, he was incorrec for a flood sufficient to cause nuisance to riparian owners sure sufficed to provide the "healthy scour" that that critic demande

The location of sewers had been complicated by road service but so congested were parts of the Chiswick Low Level sewer th the working shafts had had to be located in side roads. For examp under High Street, Brentford, there were twenty-one pipes a conduits for various services. The side-entrance manholes adopt were similar to that shown in Fig. 7, Plate 1. Although the sew there was 50 feet deep and had been constructed of cast-ir segments, the air-inlets had been bored so accurately that it h been found quite easy to avoid the circumferential flanges of t segments.

The low-level systems had required special investigation, f one was 9 miles long and the other was 5 miles long. It h been possible to construct the sewers as shallow as possible ever where and on the flattest permissible gradients (thus reducing t lift at the pump-well at Mogden), or alternatively, to tilt the sew steeply, thereby reducing its diameter but increasing the pumpi lift. On the West Middlesex scheme it had been estimated that would be cheaper by about £44,000 to tilt the sewers steeply, a that method had been adopted. As a result almost the whole the low-level system had been constructed in tunnel in blue cl The principle of "draw-down," referred to on p. 475, could be appl to most outfall sewers, and on the West Middlesex scheme it rep sented a saving of approximately £35,000 over what might termed usual practice.

In most of west Middlesex the water-logged ballast that almost everywhere encountered had required the use of steel she piling, and driving that piling had naturally caused many complai of noise. Driving piles around a tunnel-shaft had not caused m trouble, as the necessary piles had been driven in a few days. trench-work, however, piling had perforce been more or less c tinuous, and so had been complaints, even though driving had b limited to certain hours. Where tunnelling in compressed air become necessary in residential districts the low-pressure c pressors had caused most objection, and the complaints had b extremely difficult to deal with because people had been affecte differently and the psychological effect of noise could not be asses He had known places some distance from the compressors wh pictures had rattled on the walls of houses, whilst other ho

arer to the compressors had been unaffected. Vibration of two Mr. Braine. ads had been noticed; one kind had been transmitted through the rth and the other through the air. The regular beat of the exhaust the engine had been an anathema to some people but not to hers, and those nearest to it had not been always the worst fferers. At one time it had been thought that a corrugated-iron mpressor-house had acted like a loudspeaker, but its alteration d done little good. Owing to the unreliability of the electric pply it had been specified that compressors should be steam- or esel-driven, with an electric standby. Even in congested areas the noke from the steam-engines had caused less complaint than the ise of the diesels; on about the only occasion on which an electric andby had been called upon to work there had been a black-out, llowed by a "run" in the tunnel, but luckily no great harm had sulted.

Mr. A. J. MARTIN remarked that the policy of concentrating the Mr. Martin. wage of half a county in a single works was a new departure, but ventured to say that it would be widely followed wherever the cumstances were favourable. All would sympathize with the gineers in the necessity under which they had been placed of rrying out the design and construction of the works concurrently d, still worse, of having to reconsider their plans from time to ne in consequence of growth of population and other causes. In ose circumstances the completion of the works so near the eduled time was a splendid achievement and represented a umph of organization.

He was very glad to see that the flow in the sewers was controlled means of orifices rather than weirs. In view of Mr. J. D. Watson's ry successful work at Birmingham he would not have been rprised if the Mogden works had been designed on the same lines: at was, with sedimentation-tanks, a partial treatment with ivated sludge, and filters to complete the work. That, however, d not been done for the reasons given in the Paper, with which thought all would agree. It was of interest to see that the thor's firm had used diffused air in preference to any of the thods of mechanical agitation, and their judgment was bound carry weight with engineers. Apparently they still retained an en mind with regard to the relative advantages of ridge-and-row and spiral flow in aeration-tanks. They kept the storm-ter tanks full; that was the reverse of ordinary practice, but it s impossible, on reading their reasons, to say that they had not en right in doing so. In view of American experience, the pacities of the sedimentation-tanks would seem to be generous, t no doubt the population of the county would increase, and the

Mr. Martin. surplus capacity, if any, could be used up. The arrangements dealing with the sludge were particularly well planned, and it wou be of interest to see how much power could be obtained from sludge-gas. It was 43 years since Mr. Donald Cameron had int duced the process of sludge-digestion; 2 years later Mr. Mart under Mr. Cameron's direction, had lighted the works at Exe with gas obtained from the digested sludge.

Mr. Anderson. *** Mr. R. A. T. ANDERSON pointed out that the work of administration department had been divided into two disti categories: firstly, administration and supervision of the work construction, and secondly, costs.

In 1931 there had been no organization in existence capable dealing with a scheme such as that described in the Paper, and m for supervising the work had had to be selected from all parts the country, and formed into a smoothly-running organization the minimum of time. The Engineers had been fortunate in th owing to the low ebb of employment in the profession, men of type required had been available, and there was no doubt that similar job were undertaken at the present time there would considerable difficulty in obtaining suitable men for supervisi Some of the difficulties which had been involved in the organizat in keeping the numbers of a continually-changing personnel up requirements, and of ensuring the smooth working of contra might be understood when it was realized that at the peak per of work, extending over some 6 or 8 months, some sixty-five m of whom about thirty were chartered civil engineers, had b employed in the head office and as resident engineers, as well sixty-three clerks of works and inspectors. During that time less than twenty contracts had been running simultaneously, of aggregate value of over £3,300,000. From the start it had b realized that it would be necessary to co-ordinate and standard as far as possible the methods of dealing with contractors. Ow to the speed at which work had to be carried out, it had b essential to avoid discord, and it had only been by careful organ tion that harmony had been maintained in a condition that exis at one time of one single contractor working four separate contra each under its own resident engineer.

Mention had been made in the Paper, and by certain speakers the fact that, owing to lack of time, much detail design had had proceed simultaneously with construction, but little or no idea conveyed of the enormous amount of work which had been invol

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*** This and the succeeding contributions were submitted writing.—SEC. INST. C.E.

ı the billing and pricing of thousands of items relating to that Mr. Anderson. pecial work. Little information had been available of the exact osition of the points of connexion with local sewers and in most ases provisional sums had been included in the contracts to cover ıe cost of connecting manholes. When the contract had been let, ıe position was surveyed and the manhole designed, billed and riced, the cost of that work on the whole scheme reaching a figure f not much less than £100,000.

At Mogden the alterations and extensions to provide for the ıcreased population had resulted in a volume of work which had ıreatened more than once to dislocate the organization. In addition › that work, there had been the routine work of checking monthly ;atements of accounts and issuing certificates. The average ıonthly value certified during the 4 years was over £92,000 and the ıaximum in any one month £249,000.

A number of detailed costs relating to sewers were given in the aper, but costs of the purification-works were not included. Final ;counts were not yet completed, and so many outside factors had be taken into consideration that it was difficult to arrive at ;ures of cost that might afford useful information for the future. aking allowance as far as possible for those factors, costs of indidual units, including the appropriate mechanisms and all relevant pework, were approximately as follows :—

	£	s.	d.	
Storm-water tanks	7	18	0	per 1,000 gallons capacity.
Primary sedimentation-tanks	12	7	0	
Secondary sedimentation-tanks	5	15	0	
Aeration-tanks	12	0	0	
Final separating-tanks	11	10	0	
Primary sludge-digestion tanks	9	14	0	
Secondary sludge-digestion tanks	3	3	0	
Sludge-drying beds	6*s*. 3*d*. per square yard.			
Buildings (figures supplied by the architects)	1*s*. 3*d*. to 1*s*. 4*d*. per cubic foot.			

As far as could be judged, the total cost of the Mogden and Perry ıks disposal-works would come out at about 31*s*. per head of the pulation catered for. Considering that those works were unubtedly the most comprehensive and most completely equipped the world, including as they did a self-contained power-producing ınt, complete sludge-disposal works, highly-mechanized purifican-plant, an unusually large pumping-station, a very complete tering-system, extensive workshops, and an administrative ilding, it was bound to be agreed that the results showed very ɔd value for the money.

Mr. Atkins.

Mr. M. R. Atkins wished to express his appreciation of t extremely practical nature of the scheme. An engineer was som times induced by theoretical considerations to adopt designs whi for practical reasons he would like to discard. No one, after readi the Paper, would need to hesitate in future to build circular sewe instead of egg-shaped sewers, or to adopt natural surface ventilatic

He would like to call attention to two points in the Paper whi seemed to him of special interest to engineers overseas. The fi point was the contractors' preference for tunnelling as opposed open-trench work. His own experience made him favour tunnellir but unfortunately tunnelling required specially-trained labour. Colombo European tunnel-foremen had been employed over 30 yea ago to train the labour, and the result was that sewers in that ci were still being laid successfully in tunnel, the work being done local labour under local supervision in first-class European sty In Calcutta, on the other hand, and, he believed, in Madras, contractor would attempt anything but open-trench work. T result was that sewers more than 20 feet deep were rendered ir practicable, and whenever a sewer of medium depth had to be la in a street it was a foregone conclusion that some of the gas- a water-mains and electric cables would have to be relaid, and, often as not, the street closed temporarily to traffic. He wou therefore impress on any drainage-engineer who had occasion undertake work in undeveloped country the importance of obtaini the services of European tunnel-foremen from the start.

The second point was the adoption of a new type of shield which tunnelling could be carried out in water-logged ballast witho the use of compressed air. The use of compressed air was genera impracticable in the Colonies or in India, except on very large worl and a shield of the kind described on p. 474 would appear to be general application.

In conclusion he would like to stress the special significance whi a joint scheme of the magnitude of that described in the Paper h for the town-planner. The West Middlesex Main Drainage schem ought not to be left to stand alone as a notable example of wh could be done. It ought to be one of a series of such schemes, be carried out in different parts of the country simultaneously wi the spread of regional planning. One of the greatest needs expanding cities was the need for playing fields and parks. It w his confident hope that in future many of them would be found the sites of dismantled local sewage-treatment works.

Mr. Balsom

Mr. E. V. Balsom observed that, towards the end of the Pap the Author paid a graceful tribute to the Contractors and their sta who assisted in the work. As engineer to one of those Contracto

r. Balsom would like to thank him for that kindly reference, and Mr. Balsom.
would also like to take the opportunity to express appreciation
the courteous assistance which was so readily given by the
uthor's headquarters staff and by the Resident Engineer and his
sistants. For many of the contractors and probably most of the
b-contractors, the successful accomplishment of their part of the
ork had meant the breaking of all previous records. In the case of
e plant for which Mr. Balsom had been responsible, production had
d to be increased five-fold over the previous highest rate, whilst
e time allowed for erection at site had been about one-third of that
rmally allowed. Speed by itself had not, however, been suffi-
ent; it had had to be controlled speed, regulated to keep pace with
e contractor ahead and to avoid hindering the contractor following
hind.

Mr. J. D. Watson had mentioned that there was little that was
w in the works. Whilst that might be true it was nevertheless
ually true that the engineers had adopted a bold policy and had
corporated many new and improved applications of known methods
d plant; that would be shown by the following points. With regard
the use of sludge-gas, the Mogden plant was the boldest attempt
ich had ever been made to show that a modern sewage-works
uld be self-supporting in normal power-requirements. Results
d justified that enterprise. The Author stated on p. 515 that the
oblem of the best width for air-diffusers, and of the relative advan-
ges of "spiral-flow" or "ridge-and-furrow" aeration-tanks, had
en faced by adopting both sizes of diffusers and both types of
ks. It would now be possible to settle the matter under truly
mparable conditions. With regard to the final separating-tanks,
ilst some engineers and chemists might not be able to agree with
Author that shallow swept tanks produced a better sludge than
ep-sided pyramidal tanks, all would agree that the novel design
olved for Mogden produced both effluent and sludge of unusually
h quality. With regard to the form of final separating-tanks,
r pyramidal tanks 26½ feet square, with hopper sides sloping at
degrees to the horizontal and with a total water-depth of only
eet more than the 60-foot diameter circular tanks, would provide
same capacity, the same upward velocity, and 2¼ times the
gth of peripheral weir. Alternatively, if the same four hoppers
re constructed in one tank 53 feet square, the length of the
ipheral weir would still be greater than that for the circular tank.
t was of particular interest to note that space had been left for
addition of pre-aeration tanks, because such pre-treatment,
hough originally conceived by British engineers, was unfortunately
so well-known as it deserved to be. Wherever it had been

Mr. Balsom. adopted and operated correctly it had proved to be of great benefit t the subsequent stages of treatment. The tanks were usually sma replicas of the aeration-tanks and had a capacity equal to onl from 15 to 30 minutes of the dry-weather flow. The best result were obtained when the surplus activated sludge was discharge into the sewage and gently mixed in those tanks by diffused ai Where that was done it was found that: (a) the bacterial energy in th surplus sludge, which was precisely the same as that in the activate sludge returned to the aeration-tanks, was used to the full instea of being wasted; (b) the surplus sludge, when intimately mixed wit the sewage, clotted out much of the finer matter in suspension an also part of the colloidal matter, and so increased the proportion c crude sludge arrested in the sedimentation-tanks; (c) the genti "simmering" by diffused air caused grease, etc., to be freed i greater quantities so that it rose to the surface and could be skimme from the sedimentation-tanks; (d) the cumulative effect of the for going was to reduce considerably the load on the aeration-tanks, t provide a healthier activated sludge and actually to reduce tl power-consumption.

At Mogden there was at present ample capacity in the sediment tion- and aeration-tanks, and ample power was available, but wi the continued increase of population the time would come when might be found that the addition of those small pre-aeration tan would postpone the necessity for major extensions.

Some years ago, whilst visiting a large American activated-slud plant, he had seen a form of disintegrator being tried experimentall In that case the disintegrated screenings had been discharged fro a shoot into a wheelbarrow, which had been emptied into the g channels immediately behind the screens. A considerable quanti of the small pieces had found their way right through the plant a could be seen near the water-surface of the final separating-tan and along the effluent-weirs. Nothing like that was to be seen Mogden, and there again the engineers were to be congratulated having found a satisfactory solution to the problem.

In discussing the storm-water pumps the Author indicated th hydraulic couplings were not fitted between the engine-flywheels a the bevel-gears. He would have thought that the very good reaso given by the Author for incorporating fluid-flywheels in the driv for the turbo-blowers would apply also to those pump-drives. W the drive for the pumps less severe on the gears? Was it possib or necessary, to disconnect the drive when starting the storm-wa pump-engines? The rated power of the engines for the storm-wa pumps appeared (p. 542) to be proportionately less than the rat power of the variable-speed electric motors for dry-weather-fl

umps. The ratio of maximum rate of pumping was 3·8 to 1, Mr. Balsom. hereas the ratio of the power of the engines to the motors was 1 to 1. Was that difference due to higher efficiency in the larger umps or to larger margins of power for the electric motors, or was due merely to a lower head ?

In the description of the gas-control chamber it was stated that ame-traps were installed. He had always considered such traps very desirable precaution, but recently, when discussing the atter with a gas engineer, the latter had discounted their value by ying that the risk was small, and that should air get into the gas stem and become ignited no trap known would stop the flame. would be of interest to have the views of other engineers on that nportant point. It was noted that purifiers for the gas were not entioned. Their use in Great Britain did not appear to be general, hereas in the earlier German plants they were adopted, apparently r the purpose of protecting meters, control-valves and similar pparatus. Were purifiers installed at Mogden, and if not, could he ke it that more extended experience indicated that they were nnecessary ?

Mr. J. W. FLANAGAN pointed out that, in the case of the bricks Mr. Flanagan. sed, it did not appear to be clear exactly what the immersion-test ad been. An absorption of 2 per cent. was specified on 24 hours. id that mean on the whole brick with the skin intact ? Recently e had had to deal with a specification which required the bricks be tested with two broken ends and boiled for 2 hours, the bsorption being specified as 2 per cent. That requirement seemed uite ludicrous, as under no conditions could that test be in any ay comparable with working conditions. Further, in his view the ndency of both strength and absorption tests now was to be much o high ; that tendency was to be deprecated, because such specifi-tions inevitably increased the cost and also the difficulty of btaining the required bricks. A highly-vitrified surface was no oubt needed as a protection against abrasion and chemical attack, ut it seemed to him to be in danger of being forgotten that the eeting was essentially only a lining, particularly when what might e called the structure proper was of cast iron or reinforced concrete. gain, even if such bricks were considered to be essential, similar marks applied to the cement-mortar and, where it was used, the ncrete backing (whether reinforced or not). Did the Author nsider that it was probable that, under ordinary construction-nditions in those materials, an absorption of anything approaching per cent. would be realized ?

It was stated that pipes up to 3 feet 9 inches diameter had been ade of aluminous-cement concrete. If that were desirable, why

Mr. Flanagan. was it not also desirable to use mortar of that kind throughout The cutwaters at junction-chambers had been formed from aluminou cement concrete. That was a very interesting construction, pa ticularly as it was understood that there was considerable difficult in getting a really good bond between that material and Portlan cement concrete or mortar. If, as was presumably the case, th difficulty had been overcome, the question at once arose as to wh the extensive use of aluminous cement had not also been preferre at Mogden and Perry Oaks in the form of a sewer-lining. The arg ment of cost would be raised, but assuming that a good bond cou be secured, would the interest on that extra cost have exceeded t Author's estimates for maintenance, taken in both cases over t loan period ?

The Author stated that he had recently seen a shield-driven tunn satisfactorily built with concrete segments of similar type to tho described in the Paper. It would be of interest to know how th construction compared with cast iron for cost and progress. T Author's experience with small shields was confirmed by ma others ; in one case known to Mr. Flanagan, the contractors dro a shield for 8-foot 6-inch external diameter iron around a 75-fo radius curve about 100 feet long, and about 25 per cent. of the rin had had to be rebuilt. Could an illustration of the Hunt-Kearn shield be given ? It was apparently intended for use without co pressed air in wet ground. Presumably the head of water would ha to be very limited in really wet ground as it would seem that otherw the amount and cost of pumping would be formidable. What d the Author consider to be the limiting head, and would that shie be suitable for use in the neighbourhood of, or passing under, hea buildings ?

Iron tunnels in wet ground appeared to be 22 times drier th those built with concrete segments in dry ground. That would see to mean that in wet ground the concrete-segment constructi would, relatively, result in a very wet tunnel. No figures we given for brick ring-work ; that information would be valuable. was also noted that, in the figures given on p. 489, the progress of 6-foot diameter iron tunnel driven without pocketing was so 50 per cent. greater than that of a 6-foot 6-inch diameter concret segment tunnel. It was not quite clear whether the iron tunn was in free or compressed air, but the result seemed surprising a the Author's views on the reason would be interesting.

His own experience of bituminous packings was similar to that s out in the Paper, but he would go much further and would say th they should never be used alone. They were much more delica than wood, and consequently were often damaged either before

uring erection, and, squeezing out as they did under pressure, leaks Mr. Flanagan.
evitably resulted. Again, it was extremely difficult to cut them
ıck to caulk satisfactorily at the points where leaks occurred.

He was very interested to learn that the Author had been able
make arrangements to pay a bonus of 10 per cent. on certain
ɔrk. He thought that it was much to be regretted that that could
ɔt be done on all public works completed before the date specified,
ovided always that a comparable penalty was enforced when
ɔrk was behind time.

The history of the construction of the pump-well at the Mogden
ımping-station was curious. Surely in work of that sort move-
ent within limits was bound to be expected and allowance made
r it. Why could not the incoming low-level sewers have been
iven up to a point at some distance from the well, and then con-
ected up, if necessary by a flexible-joint construction, after the well
ıd been finished? The example of Mogden was valuable and the
uthor was to be congratulated upon setting it out in the Paper,
ecause it ought to be recognized as widely as possible that move-
ent in deep excavation or in tunnels, although it might (and
ould) only be small, was bound to take place. In so far as the
nbering of the hole was concerned it would be helpful to know
hether the boards were middled, tucked and poled, or piled. In
ı excavation of that magnitude the 5-foot poling boards seemed
ery long, and if on account of the reinforcing they had had to be
ed, that would only emphasize the undesirability of allowing the
w-level sewers to be driven so close up to the well.

With regard to the plant, it would be more useful if the bucket-
pacity were stated rather than the capacity per shift; the latter
as necessarily a very variable quantity.

The degree of purification which was now obtained was amazing;
, however, it was understood that there was now a considerable
nount of dilution, it would be useful to know what was expected
hen the sewers were under full normal load. It would also be
ery useful to have a gradient of purification through the various
ages of the works; that was, taking the suspended solids and the
ssolved oxygen at 65° F. in 5 days of the flow at the following
ints: before screening, after screening, after removing the
it, after primary sedimentation, after secondary sedimentation,
ter aeration, and the effluent. If that could be done under con-
tions of steady flow and as nearly simultaneously as possible,
most useful diagram of the effect of the various stages would be
esented.

In view of the wonderful results obtained it would be as well to
ıow exactly to what extent the pollution of the river Thames had

Mr. Flanagan. been reduced, if possible on the same basis as the effluent-requir ments. It seemed to him that the results should be made as wide known as possible, through the public press or otherwise, in ord that the public should not be influenced by irresponsible and mi leading "opinions" regarding the alleged pollution of the Thames

Mr. Graham. Mr. D. C. GRAHAM observed that, although Sir George Humphre and Mr. Hetherington had both pointed out that the West Middles Main Drainage scheme was a concentration of a large number smaller sewage-works into one large works, as recommended in t report on Greater London Drainage issued by the Ministry of Heal in December, 1934, neither of them had drawn attention to the fa that in one important point it differed from the recommendation that was the inclusion in the scheme of a portion of the watersh of the river Colne which happened to be within the county Middlesex. It would appear that the sewage-works in that portio of the Colne watershed might well have been left until such time they could be included in a main-drainage scheme for that watershe and the substantial cost of conveying the sewage from 10 to 12 mil across the county of Middlesex to Mogden could thus be avoide Possibly as a result of that drainage being so diverted, a scheme w now before Parliament for the main drainage of the upper part of t Colne watershed in the county of Hertford ; no doubt another schem for the lower portion in the county of Buckingham would be broug forward in due course, with the result that the sewage of that ar would be dealt with at four sewage-works instead of one as suggest in the report of the Ministry. The Middlesex scheme had bee decided upon before the Ministry's report was published ; there we no doubt reasons for the scheme adopted, but it was an example the effect of carrying out drainage-schemes for local-governme areas and not watersheds.

The Paper showed the great advantages a scheme of that kin carried out under special Parliamentary powers in an area co paratively free from obstructions below ground and only develop in part, had over a scheme carried out after the area had been ful developed, with its pipes and services, subways, railways, cuttin and tunnels and other obstructions. Attention had been drawn the obstructions in Chiswick High Street, but in a fully-develop area those were found in every main street, necessitating worki shafts in side streets and expensive diversions of pipes to get ev room for manholes and entrance-shafts. Another great advanta was the very small number (one hundred and thirty) of connexio to be provided for and the fixed quantity of flow to be carried, bo of which greatly simplified the design of the scheme.

Would the Author add a larger plan showing the drainage-area

e sewers with skeleton longitudinal sections of the sewers, giving Mr. Graham.
ies, levels and gradients ?

The "McAlpine" sewer-section was of great interest. It had parently not been adopted on account of any saving in cost, and e of its chief advantages appeared to be that the percentage of en from the Distressed Areas employed would be higher, and so that the limited supply of sewer-bricklayers would not be lled upon to the same extent. In view of the latter reason, it as rather surprising to find that both in that section and where ey were constructed with cast-iron segments, the sewers were tirely lined with brickwork. In London, for the past 30 years or ore, sewers made of cast-iron segments had been extensively nployed, but only the invert had been faced with one ring of ickwork, the remainder of the sewer being formed of good-ality concrete. That type of construction had been found to entirely satisfactory in practice. The bolts in the cast-iron gments formed an excellent key for the concrete, and there would t appear to have been any difficulty in providing a suitable key r a concrete lining in the "McAlpine" segments. In several rge sewers in London which had been lined throughout with one ng of brickwork and concrete backing, it had been found that in urse of time the brickwork separated from the concrete, became drummy," and finally became unsafe. The initial cause appeared be the gradual seepage of water through the concrete and its llecting behind the brickwork; could the Author say if any movement of that kind had been noticed in the West Middlesex sewers ? typical sections of the sewers used were added to the diagrams, they ould be of considerable interest.

It was not clear from the description whether the one ring of ickwork lining had been bedded directly on the "McAlpine" gments and, if so, how the line and level of the sewers had been aintained. It was stated on p. 473 that "It was found possible to ect the segments with a tolerance of ½ inch in line and level." uld the Author state whether the sewers were constructed with at small tolerance throughout ? In the cast-iron sewers in London ready referred to, it had been found advisable to set the inner ing eccentric to the cast-iron ring, and by that means to obtain a erance of from 1½ to 2 inches. A true line and level could thus be aintained without "cropping" the bricks. The description of Mr. nt's shield was of interest, but it was not easy to understand thout a diagram. Particulars of the cost of driving with that ield through waterlogged ballast would be of value.

Certain districts were drained on the combined system, and others the separate system, and in some cases there were large quantities

Mr. Graham. of subsoil water leaking into the sewers. As it appeared that the was capacity in the sewers for 240 gallons per head of populati per day, and since provision was made for gauging the flows, would be useful if it could be stated how that quantity was shar between the different districts. Those on the completely separa system could not use the capacity of 240 gallons per head per d provided, whilst those on the combined system would require it a and those with much subsoil-leakage would have the advantage taking six times the sewage-figure of 40 gallons per head to the worl It was presumed that in all cases the payment made for dealing wi the sewage was based on the rateable value of the district.

In many of the local sewage-works, large tanks had been co structed in the past for treating the sewage; had the use of the tanks for storm-water settlement, and the resulting saving in co been considered before it had been decided to carry such a lar quantity as six times the dry-weather flow so many miles to passed through similar tanks at Mogden? By using the existi tanks and, after the storm was over, passing the contents of th into main sewers, it would appear that a large expenditure in tanks and on sewers might have been saved. That method had be found satisfactory in many places and had the further advantage retaining for the streams a considerable portion of their natu flow. The object of screening the sewage through a $\frac{3}{4}$-inch spaci was not clear. It had been the policy of the London County Coun to reduce screening to that necessary to protect pumps and prev choking of pipes. Very considerable savings had been effected removing fine screens. The use of wood packing coated w bitumen was of interest; the efforts of the manufacturers to prod a bitumen packing had been disappointing. In certain cases, wh the samples submitted had proved satisfactory under test, eq quality could not be supplied in bulk.

The vertical outlets under the bed of the Thames were of espe interest. In spite of the upward velocity, it would seem likely t heavy material travelling along the bed of the river would drop i the vertical pipes. If those outlets had been laid off since they l been brought into use, would the Author give an account of th condition?

Mr. Hillier. Mr. W. H. Hillier admired the ingenuity shown in ensuring t the average velocity of flow through the grit-channels was 1 f per second at all times. He pointed out, however, that it important to obtain a good distribution of velocity over the cr section of flow. Were steps taken in design to achieve t and to reduce eddy-formation to a minimum? From the figu given he calculated that the velocity in the suction-pipe of the pu

edger was about 13 feet per second, and he wondered if that was Mr. Hillier.
fficient to remove all the grit deposited in the channels. In actice some sludge settled with the grit, and, unless removed fairly ickly, the mixture "packed" and was then difficult to remove.

He thought that the washing action in the pump-dredger, together th the subsequent washing in the upward-flow settling-tanks, ould result in a grit sufficiently clean to tip. It was generally cognized that a grit which contained less than 10 per cent. of latile organic matter could be tipped without causing nuisance. seemed to him that the final washing of the grit in the settling-nks with river-water was unnecessary. He was interested in the ethod adopted for removing the ballast from the site of the rification-works, and wished to know the velocity required in the imp-suction.

As the screen-raking mechanism was operated through the agency differential air-pressure, were the air-pipes fitted with diaphragms was the air-reaction method, in which air was slowly bubbled into e sewage, adopted? The Author stated that "Prompt removal sludge from sedimentation-tanks is a feature of modern practice," it Mr. Hillier understood that only one scraper was provided for e secondary sedimentation-tanks. From the travel-speeds given calculated that the net time required to remove the sludge from ose tanks was 10 working-hours, no allowance being made for ansportation, etc. In practice that would probably mean that the nks were sludged once every 2 days, which in his opinion was not ten enough. Sludge ought not to remain in contact with sewage r more than 24 hours, particularly in warm weather.

Fig. 18 (p. 503) indicated that surplus activated sludge was added the sewage upstream of the primary sedimentation-tanks. He ould have thought it preferable to add the sludge to the effluent om those tanks, so that the flocculating properties of the sludge ould not be partly wasted on suspended matter that would settle adily. He was surprised that pre-aeration tanks had not been ovided between the primary and secondary sedimentation-tanks. short period of pre-aeration in the presence of surplus activated idge had been found to result in a definite improvement in the luent from secondary sedimentation-tanks at other works. The ovision that final effluent could be returned to the aeration-tanks s to be recommended. Elsewhere that had proved very beneficial ien strong sewage was being dealt with, for the dissolved and mbined oxygen in the returned effluent had increased the purifica-n in the aeration-tanks and had improved the condition of the ivated sludge. If the returned effluent contained a relatively ge quantity of nitrates, as the Mogden effluent probably did,

Mr. Hillier. there was a danger that nitrogen would be liberated and woul prevent efficient settlement of sludge in the final separating-tanks.

The reasons given for fixing the diameter of the final separating tanks at 60 feet did not seem conclusive. Once the required velocit of upward flow in the tanks was fixed, the total surface-area require was readily obtained, but tanks having diameters other tha 60 feet would seem to satisfy the other conditions stated. A unusual feature of those tanks was the provision of scrapers for th vertical walls. It was usually considered that walls inclined at a angle of not less than 60 degrees to the horizontal did not requi scraping. It would be of interest to know the detention-period o dry-weather flow in the final tanks.

It was expected that the form of impeller for the return-sludg pumps would obviate risk of damage to the sludge-flocs. Son details of the impeller and of the limiting peripheral speed wou be of interest.

The unstirred sludge-digestion tanks at Mogden were provide with fixed draw-off pipes for separated water, but it would be ve difficult to tap liquor in a primary digestion-tank by such mean He would have thought that provision should have been made f treating separated liquor before discharge into the aeration-plan At some works separated liquor had caused considerable trouble biological plants, and it was common practice to provide means f aeration and cooling. Some form of storage was advisable so th the rate of admission of liquor to the aeration-tanks could l regulated.

It would be of interest if the Author could give some details the sludge-meters. Where venturi-meters of the standard ty were used, special care ought to be taken to prevent the entry sludge into the pressure-piping and float-chambers of the record Orifice-meters had been used with success and the air-reacti system had been recently employed. Could the Author say wh coefficient of friction had been adopted in calculating the fricti head in the long sludge-pumping mains running from Mogden Perry Oaks, and had any tests been made to verify the figure chosen

At Perry Oaks steel sheet-piling had been driven through 12 fe of ballast. Had difficulty been experienced in driving, and ha such special means as water-jets been required ?

A further Paper dealing with the operation of the plant and t results of the experimental work referred to by the Author woul he thought, be of great interest to those concerned with the desi and operation of purification-works.

Mr. Hogg. Mr. Charles Hogg observed that reference was made on p. 494 the strengthening of the effluent-conduits to resist internal pressu

ıe design and construction of the effluent-conduits was divided Mr. Hogg
to two parts, and the work shown in Fig. 14, Plate 1, in the imme-
ate vicinity of the river Thames, had been executed in advance
the connecting tunnels between Mogden and the shafts A and B.
had originally been thought that there could be no excess of
ternal pressure over external pressure at that point, since the con-
uits had free discharges to the river, and that the water-level in the
ver would also control the external pressure on the tunnel-lining.
When the tunnels were being driven, however, it had been
served that the face had been quite dry, and the matter had been
considered. Calculations had showed that the lining was suffi-
ently strong to resist the internal pressure, without any counter-
lancing external pressure, if it were assumed that the circum-
rential flanges of the iron acted as hoops, with the skin spanning
tween the flanges. The joints in the circumferential flanges were
h-plated by the adjoining flange, owing to the specified "reeling"
the iron. The pressure of the bituminous packings, referred to
p. 494, had, however, introduced a doubt as to whether the
sumed conditions would obtain in practice, and it was felt that
cessive bending stresses might be developed at the roots of the
nges. It had been decided to web the flanges on the section of
nduit remote from the river, but it was then too late to alter the
on on the river section, and had it been necessary to strengthen
e lining there, the only course open would have been to reduce the
ective diameter of the conduits slightly, and to embody reinforce-
ent in the concrete lining.

It was therefore decided to test a short length of tunnel-lining
situ, with pneumatic pressure. An improvised measuring-
vice was erected in the air-lock to measure the extension of
e horizontal and vertical diameters under pressure, the exten-
n being magnified 24 times and shown by a pointer on a scale
ich could be read from the outside through the glass peep-hole
the lock. The pressure was then raised in the lock to 20 lbs.
r square inch, which was greater than the maximum hydraulic
ad in service, measured at the invert of the tunnel. The extension
diameter amounted to 0·017 inch. Owing to possible errors of
servation, precise stress calculations could not safely be based on
at result, but the significant fact emerged that both pointers
turned to zero on decompression, showing that the movement was
stic. The tests were carried out at low tide, so as to make the
nditions as severe as possible.

The iron of that shield-driven tunnel had previously been grouted
ternally to a pressure of 70 lbs. per square inch, but it was
ident that the passive resistance of the surrounding clay could not

Mr. Hogg. be relied upon, when considering the effect of internal bursti pressure. The tests had been carried out on the bare iron, whi had subsequently been lined with concrete and one ring of brickwor no action had therefore been taken to depart from the origi design. After some months in service, both conduits had be emptied and inspected, and had been found to be in perfect conditio The results of those tests agreed with the conclusions reached Mr. Gerald Haskins, in his Paper on the Sydney Water Supply.[1]

Another interesting feature was that it had been possible, ev at that early stage, to produce artificially the maximum-flow co ditions for which the effluent-conduits had been designed; it h been found, after making certain assumptions, considered to reasonable for losses at inlet and exit, that the value of n=0·013 Flynn's modification of Kutter's formula, referred to on p. 471, w substantially correct.

The Author. The AUTHOR, in reply, observed that the terms under which t County Council accepted the sewage of local authorities explai why the Council had had no choice in the construction, and lat in the maintenance, of the small sewers referred to by Sir Geo Humphreys. The Council's obligation was to accept the sewa of local authorities at specific points (that was, usually at the fo point of local sewerage-systems), and except where special arran ments without the Act were made, the maintenance of the sewe irrespective of their size, was borne by the County Council. T figure of 60,000 yards of sewer of 3 feet diameter or less quoted Sir George was, however, somewhat misleading, because about miles of that total was made up of duplicate sludge-mains and o the remaining 37,000 yards represented sewers of 3 feet diamete less. All the sewer-sizes given in Appendix I (pp. 558 to 565) refe to the internal diameter of the finished sewer. That was to s the outside diameter of cast-iron segments, although varying so what, was about 2 feet more than the internal diameter of the se as given in the third column of Appendix I.

The description of some of the special backdrops referred to Mr. Temple was treated in the Paper as only an illustration of c struction, but it was unfortunately not stated that some of illustrations chosen were of backdrops used solely for surface-wa connexions. Mr. Manning had gone into that more fully in remarks.

In speaking of the length of the grit-channels, Mr. Temple see to omit consideration of the depth of flow. Those channels w

[1] "The Construction, Testing and Strengthening of a Pressure Tunnel fo Water Supply of Sydney, N.S.W." Minutes of Proceedings Inst. C.E., vol. (1931–32, Part 2), p. 25.

-signed for a horizontal velocity of 1 foot per second, and for a vertical velocity of 0·1 foot per second for the settlement of grit. That was to say, unless the length of the grit channel was ten times the depth of flow, there might still be grit left in the sewage. Since the depth of flow could be rather more than 9 feet, the length of the grit-channel had to be at least 90 feet without allowance for inlets and outlets. The Author.

An answer to Mr. Peirson Frank's query about return-sludge volumes was to be found in Mr. Whitehead's remarks. Mr. Hetherington drew attention to the steadying influence on the strength of the composite sewage owing to mixing the sewages of a number of districts. Whilst that was undoubtedly the case, the degree to which it could be depended upon, and the constancy of that condition, were not necessarily reliable, so that allowance was made for controlling the volume of return-sludge within wide limits and for diluting the untreated liquor. That was only prudent; more especially was it reasonable when the layout lent itself to the arrangement.

With reference to Mr. Frank's remarks, the Government Grant was not computed on the whole cost of the scheme, but on £4,290,000, as stated on p. 465, and the County Council had to find a sum of ,000,000. Mr. Morgan gave particulars of rates in his remarks, and showed that without the grant the rate would be 10·03*d.* in the pound, instead of 1*s.* 4*d.* If the Middlesex sludge had been taken out to sea it would have been costly to the county. Not only was the quantity per head much in excess of the London figures owing to the nature of the purification effected, but the barges would have to be much smaller to navigate the river, they would be dependent on tides, and they would take twice as long to complete a trip. The combined effect had been estimated to cost 1*s.* 0*d.* per ton, whereas the present arrangements were estimated to cost 6*d.* per ton without crediting the value of the power obtained from the sludge-gas. Mr. Townend had showed that the value of the gas was such that the sludge-disposal costs were completely nullified, and it was therefore correct to say that sending the Middlesex sludge out to sea would have cost the County Council 1*s.* 0*d.* per ton, whilst they would, in addition, have had to spend £20,000 per annum on power, which at present was saved. It was scarcely necessary to mention the objections that would be raised to taking sludge-barges through London.

It was not yet 12 months since the whole flow had been discharged to the Mogden works, and costs of treatment could not yet be available. The overall cost of running the Mogden works as estimated for the financial year 1937–8 had, however, been given by Mr. Morgan.

The Author. On p. 516 an air-supply of 2 cubic feet of free air per gallon of sewag based on the dry-weather flow, was given as a maximum, and c p. 517 it was stated that 1·4 cubic foot per gallon was the anticipat normal figure. The latter figure had not yet been attained, 1 cubic foot having so far been the average, but the large volum which were being treated affected the figure. During the whole one month in the present year, for instance, the average daily flo was $2\frac{1}{2}$ times the dry-weather flow.

Mr. Frank had suggested that the main contract might have bee sufficiently speeded up by a value—cost contract to enable the co tractors to construct supplemental work as well as the main contra but he did not appear to have fully appreciated the acute congesti on the Mogden site during the busy years. The main contract that case already demanded of the contractors that they should, wi too small an area for communications, stores, and workshops, pe form their work at a speed rarely attained. In asking them undertake the supplemental work as well, the Council were virtua asking them to sacrifice that area of land which would have be invaluable to them and to construct more works on it, leaving tota indequate working space for yards, etc. The contractors were, fact, being asked to do more than seemed possible in the tim their risks were already considerable and their difficulties seemed times insurmountable, so that to induce them to accept an ev greater risk it was necessary to offer them not only greater remune tion but a higher rate of payment. Without it, it was possible th even the main contract would not have been finished in the tin and that no Government Grant would have been earned by t Council on the supplemental work. The Author believed that t small amount of the bonus, payable as it was on only the supp mental work, was saved many times over by the Council. T remarks on p. 499 explained why the available site had been diminis by the ballast-dump before the main contract had been let; no fo that the main contract could have taken would have affected th

Figures to show the effect of storm-water discharged by constituent districts would no doubt be available in the course time to answer the question raised by Mr. Hetherington, but expectation was justified that the variation in the rate of flow Mogden was not so noticeable as the similar effect on a sma system. Experience of operating conditions obtained by Townend, and mentioned by him and by Mr. Braine, went to sh that that had already been established as a fact, even if the s porting data were as yet meagre.

Although Dr. Calvert was correct in saying that there had been need to make the design of the Mogden works fit into some alre

xisting works, which would unquestionably have complicated the The Author. esign, it was worth emphasizing that the layout had had to be so lanned that the old Heston and Isleworth pumping-station could e retained in commission on the site until the new works were in peration, and that the sewage from a population of 100,000 persons ad had to be treated on the site continuously, old tanks and contact-eds having been demolished only when new storm-water tanks were eady to replace them, and when temporary sludge-digestion lagoons nd drying-beds had been made on the already too-small site available or working. In that respect it was incorrect to suggest that designers nd contractors had had a free hand on a virgin site. The additional easons for effecting sedimentation in two stages were found in the rst paragraph on p. 511. Mr. Martin had expressed the opinion that, 1 view of American practice, the capacity of the sedimentation-tanks eemed to be generous. Whilst that was so, a comparison with inglish practice would seem to suggest that the capacity was smaller 1an was customary. The smaller the tank-capacity the less pro-ection did the biological-treatment plant have from sudden changes 1 strength of the crude sewage. It was assumed, as Mr. Hethering-on pointed out, that those sudden changes would be less noticeable t Mogden than in smaller systems, but even so it would be unwise o reduce the sedimentation-tank capacity unduly, because the -hours capacity provided was not the bare essential arrived at as the esult of consideration of only those theoretical conditions governing he settling of solids in a tank, but also took into account the desira-ility of affording the management time to observe and act on any nusual change in the strength of the sewage arriving. Tankage of easonably large capacity was the only means of controlling such hanges.

A positively-scraped tank was preferred for the design of the nal settlement-tanks rather than the design suggested by Mr. Balsom. The depth of flow over the weirs, even at the rate of three mes the dry-weather flow, was rather less than $\frac{1}{2}$ inch, and it was ot easy to see what would be gained by a still thinner film. Mr. Balsom was possibly correct when he pointed out that the addition f a pre-aeration process might become necessary in time, and on . 511 it was stated that space had been left for just such auxiliary lant.

The compressors were geared up to run at 10 times engine-speed, he normal speed of the engines being 300 revolutions per minute. Very large volumes of compressed air would have been necessary o rotate the heavy mass of the impellers of the compressors at 10 mes the firing speed of the engine for starting, and the hydraulic ouplings not only smoothed out the torque-fluctuations but allowed

The Author. the engines to be turned over at firing speed before any appreciabl torque was transmitted to the impeller shaft. The storm-wate pumps were only geared up at 1·56 to 1 from the engines, and n great torque was therefore required to turn the engine over at firin speed, so that it was unnecessary to disconnect the drive whe starting them or to provide a hydraulic coupling. The power-rati of the engines and the electric motors was due to the higher efficienc of the larger (storm-water) pumps. No gas-purifiers were fitted a Mogden, and so far experience had not shown that they wer necessary.

Mr. Flanagan referred to the absorption-tests applied to the brick but all tests that were applied to bricks were aimed at obtaining choice of only that class of brick which was likely to be acceptabl for the work. It was not thought that the absorption-test wa necessary for its own sake because the working conditions wer different from the test-conditions, but a lower standard specifie for that test would automatically widen the choice of makes of brick submitted by contractors, whereas only the best brick was wante The specified test was carried out on whole bricks and there wa never any difficulty in obtaining supplies of suitable bricks for th work.

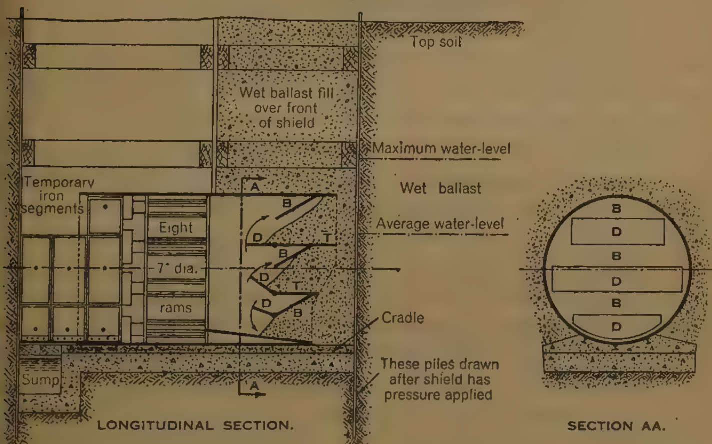
The use of aluminous cement for concrete in sewage was mo desirable and necessary in confined space such as in sewers than o sewage works. Concrete-pipe sewers could be renewed at conside able cost but in most cases they could not be repaired, and it was no therefore, the cost of maintenance of Portland cement in conta with sewage in a confined space, so much as the cost of renewal the whole of the work, which had to be taken into consideratio No difficulty was experienced in getting a good bond between Por land and aluminous-cement concretes and mortars; trouble wou no doubt arise unless the Portland cement were allowed to set f 7 days before aluminous cement was placed on it, or (in the rever process) unless the aluminous cement were allowed to set for days.

In mentioning on p. 473 that he had seen a concrete-segment tunn driven with a shield, the Author referred to experimental work onl shown him by the courtesy of Sir Robert McAlpine & Sons (Londo Ltd. He was therefore unable to give any details of the cost progress. A diagram of the Hunt-Kearney shield was shown *Figs. 46*: the patent was now held by Messrs. Markham & Compan Ltd., shield-makers.[1] It was designed for continuous excavation a full face of wet ballast without clay-pocketing or timbering, t

[1] Patent No. P. ACK. 4. 33261.

round being excavated in three chambers ranged one above the other. The ballast lay on tables T, and was caused to take up an artificially high angle of repose by means of fixed shutters B (sloping at an angle of about 30 degrees to the horizontal), and irregular oning of the compartments caused compression of the ballast. The openings through which the excavation was done were controlled by doors D. *Figs. 46* showed the method by which the shield was driven out of the pit, it being necessary to fill in front with ballast and to tighten it up to the sheet-piling by applying pressure to the rams, which was maintained while the piles were withdrawn. Normal costs of operation could not be given as the three tunnels were short, and various experiments, together with the cost of erect- The Author.

Figs. 46.



g and dismantling the shield three times, complicated the figures r comparison. In 10 hours, however, on one occasion three rings 5-foot 8-inch internal diameter cast-iron tunnel were constructed ith that shield, and it might be assumed that the cost of driving as the same as the cost of driving a shield in free air in good ground, us the cost of pumping and some allowance for slightly reduced ogress. The progress was not, however, apparently reduced to e speed of progress in compressed air under similar conditions, and e whole of the cost of the compressed air was saved as well as the cost timber, clay-pocketing, etc. The maximum head of water under ich the shield could economically be worked would depend not ly on the nature of the ballast but on the diameter of the shield, e head against which pumping had to be done from the sump, the gth of the drive, etc. A point worthy of notice was that infiltra- n was apparently considerably reduced by compression of the

The Author. ballast in front of the shield. The question whether or not th shield was suitable for use near or under heavy buildings seemed t depend entirely on the local conditions, but the risk of a "run" o ballast into the tunnel seemed to be negligible, because in no cas had subsidence occurred at either the railway crossings or the cana crossing. For the three sections for which the shield had bee designed the cost of installing compressed-air plant and drivin under compressed air had been estimated at an approximate extr sum of £22,000. The cost of the work as carried out with the specia shield had been about £12,000, inclusive of all costs of pumping provision of the special shield, experiments and day-work.

Cast-iron-segment sewers, properly caulked, ought to be wate tight even in wet ground, and it was obviously misleading to com pare infiltration rates. Any comparison between the rates of pro gress of a 6-foot 6-inch diameter concrete-segment sewer and 6-foot diameter cast-iron-segment sewer had to take into considera tion the fact that the rate of progress of the concrete-segment sew was governed by the grouting. The strength of the concrete rin did not develop until the grout surrounding the reinforcing bars i the joints had set hard, and for that reason a limit was placed on th number of rings erected during each shift. With cast-iron segmen that reason for delay did not arise, but a concrete lining was neede before it was ready to be lined with brickwork, whereas that inte mediate operation was eliminated in the concrete-segment sewer.

Owing to the speed at which all the work had to be done there w a choice only of driving the tunnels short of the pump-well at Mogde or of driving them right up and risking repairs later. The fir would have involved disorganization of the work on the Mogden sit and delay in construction of the pump-well—a vital part of t scheme.

In reply to Mr. Graham, no movement of the brickwork lining sewers had been noticed, and no trouble had been experienced wi seepage behind the brickwork. Brick lining was placed against t concrete segments with only a ½-inch collar-joint, the ½-inch toleran mentioned in connexion with the erection of concrete segmen having proved quite adequate on most of the contracts, although first it had been difficult to maintain before the miners beca thoroughly used to the work.

It would probably be some time before it was possible to gi a reliable statement of the use actually being made of the sew by individual local authorities. Although the County Coun might limit the rate of discharge from the local authorities' sew to 240 gallons per head per day, up to that rate the local authorit had a perfectly free hand, so that districts which had in existenc

eparate system might discharge at least a part of their surface- The Author. vater into the County Council's trunk sewers, subject only to the 40 gallons per head per day limit. The use of storm-water tanks t the old sewage-works of local authorities had been considered, ut was contrary to the whole principle of abolition of sewage-works n the county, and would have led to undesirable complications in ealing locally with sludge, and in the control of flows. It would ertainly have resulted in considerably greater volumes of partially-reated liquid being discharged to the streams, and the latter would ave derived no benefit since they were already over-taxed in times f rain. The effluent outlets to the Thames were scoured out egularly every 2 or 3 months, as described on p. 497. Discoloration t the time of the scour implied that river-sand did find its way into he outfalls, but the cleaning out by scouring had in the past proved uccessful in getting rid of the accumulation.

The point raised by Mr. Hillier about the avoidance of eddy-ormation had been considered not only in the design of the grit-hannels but in the design of the approach-channels. The grit emoved from the channels was of a high order of cleanliness and here was no difficulty experienced in dredging. A high standard f cleanliness of the grit was necessary at Mogden, although dmittedly there were many sewage-works sites where a much lower andard would suffice.

The air-pipes for automatic screen-operation were connected direct o a differential pressure-gauge with provision for replenishing the ir in the system periodically by means of a hand-pump. There as no continuous flow of air. Final separating-tanks larger than hose installed introduced an element of risk in delaying the removal f active sludge from the floors.

The detention-period in those tanks was 6 hours of the dry-weather ow. The pump installed for the pumping of the return sludge was mixture of the centrifugal- and axial-flow type, with a three-bladed npeller. It was made by Worthington-Simpson, Ltd., and was alled the "Mixflo" pump. It was the practice with venturi-type eters to let clean water (effluent) pass slowly back continuously om the float-chambers of the sludge-meters, and air-reaction had so been found to be a sound method, but where orifice-type meters ere employed with sludge, trouble might be experienced owing to as-accumulation unless the tappings of the reaction-connexions ere lowered to the horizontal diameter.

Very rough figures showed the friction-head of the Mogden to erry Oaks sludge-pumping mains to be only 40 per cent. in excess what would be expected with clean water.

ORDINARY MEETING.

1 March, 1937.

Sir ALEXANDER GIBB, G.B.E., C.B., F.R.S., President, in the Chair.

The discussion on Mr. Watson's Paper on "West Middlesex Main Drainage" was continued and concluded.

*** The Correspondence on the foregoing Paper will be published in the Institution Journal for October, 1937.—Sec. Inst. C.E.

ORDINARY MEETING.

9 March, 1937.

WILLIAM JAMES EAMES BINNIE, M.A., Vice-President, in the Chair.

The Scrutineers reported that the following had been duly elected as

Members.

Gwynne Burnell Brader.
Roger Francis Williams, O.B.E.

Associate Members.

John Ackroyd, B.Sc. (*Bristol*), Stud. Inst. C.E.
Herbert Ward Barkas, Stud. Inst. C.E.
Ezekiel Baron, B.Sc. (*Witwatersrand*).
Mounsey Batten.
John Bigg, Jun., B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Valentine Carnegie, Stud. Inst. C.E.
Edgar Claxton, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Norman Reyland Maxwell Craigie, M.Sc. (*Queensland*).
Geoffrey Innes Davey, B.E. (*Sydney*).
George Frederick Robert Dickinson, B.Sc. (*Bristol*), Stud. Inst. C.E.
Saville Dorfman, B.Sc. (*Witwatersrand*).
David Hay Edie, B.Sc. (*Edin.*), Stud. Inst. C.E.
Frank Douglas Gemmell, B.Sc. (*New Zealand*).
Donald Mackenzie Hamilton, B.Sc. (*Edin.*), M.Eng. (*McGill*), Stud. Inst. C.E.
Alan Blackbourn Jones.
Harry Kaylor, B.Sc. Tech. (*Manchester*).
John Douglas Knowles, Stud. Inst. C.E.
Eric John Lake, B.Sc. (Eng.) (*Lond.*), Stud Inst. C.E.
William Alfred Wallington Lankshear, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Hugh Vernon Hughes Lock, B.A. (*Oxon.*).
Frank Stanley Maconachie.
Lewis Henry Park, B.E. (*W. Australia*).
Frederick William Potter, B.E. (*Sydney*).
Robert Francis Deans Ritchie, B.Sc., B.E. (*New Zealand*).
Leonard Melrose Craigie Robertson.
Harris Oswald Terence Scharenguivel, B.Sc. (Eng.) (*Lond.*).
David Baxter Simpson, B.Sc. (*Glas.*) Stud. Inst. C.E.
Douglas Bernhard Smith, B.E.E. (*Melb.*).
James Hood Smith, B.Sc. (*Edin.*).
Frederick Howard Stewart, B.Sc. (*New Zealand*), Stud. Inst. C.E.
Robert Stewart, Stud. Inst. C.E.
Charles Clifton Strong, B.Sc. (*Bristol*).

BRIAN PHILIP TOWNSEND, B.Sc. (*Cape Town*), Stud. Inst. C.E.
WILLIAM BROOK TURNER.
WILLIAM ERIC VARRIE, B.Sc. (*Witwatersrand*).
THOMAS CHRISTIAN WATERMEYER, B.Sc. (*Cape Town*).
GEORGE HENRY WILLE, B.Sc. (*Witwatersrand*).
BASIL WRIGLEY WILSON, B.Sc. (*Cape Town*).
BRUCE RICHARDSON WINSTONE, B.E. (*New Zealand*).
FREDERICK COLLIS WOOD, B.E. (*New Zealand*).

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

Paper No. 5103.

"Welded Joints in Pressure-Vessels."[1]

By STANLEY FABES DOREY, D.Sc., M. Inst. C.E.

TABLE OF CONTENTS.

INTRODUCTION.

'OR many years riveted joints of various designs have represented he acceptable method of joining together two or more plates which orm part of pressure-units. Since the early days of boiler construcion the gradual increase in working pressures was accompanied y a steady improvement in the design of riveted joints, and in the uality of the material required for plates and rivets. Much research vork, both theoretical and practical, was devoted to the problems f riveting. Puddled wrought iron gave place to soft mild steel, nd with the design of the double-butt-strap joint, boilermakers chieved the best type of joint possible with this method of contruction. Pressures still increased, however, and when plate hicknesses for water-tube boiler-drums became too great for efficient iveting, resort had to be made to the solid-forged steel drum.

Side by side with this development, progress was being made in he practice of welding by various methods. It was realized by iscerning minds that by means of some welding process, joints vould ultimately be made which would be more efficient than those hat could be obtained by riveting, and which would provide, in ertain circumstances, a better economic proposition than the solidorged drum.

[1] Correspondence on this Paper can be accepted until the 15th August, 1937. –SEC. INST. C.E.

The fundamental difference between a riveted and a welded joint is that one is a mechanical joint and the other is a metallurgical joint. In consequence the mind of the average engineer is predisposed in favour of the former. The proper appreciation of a welded joint requires an understanding of the principles of metallurgy, and in the absence of such understanding there is bound to be some scepticism in regard to welded joints. At the same time, it is an unfortunate fact that serious failures have occurred in the past due to the rupture of welded joints in pressure-vessels, and engineers cannot be blamed if they put their faith in a proved method of construction such as riveting which has given satisfactory results over a large number of years. In fairness to the welding industry it must, however, be said that past failures have in the main been due to one or other of the following factors :—

(1) Premature application of a welding process to pressure-vessel construction.
(2) An absence of specified conditions governing details of manufacture.
(3) An absence of any adequate procedure for testing and inspecting the work.

These three primary causes of failure have been eliminated, so far as forge- and fusion-welding is concerned, and such progress has been made on sound scientific lines that the possibilities of failure of welded joints made in accordance with authoritative regulations are not greater than with the best type of riveted joint.

Comparison of Efficiencies of Riveted and Welded Joints.

The theoretical and practical analyses of a riveted joint comprise a simple problem having a solution which is readily understood by the engineer. A riveted joint can only fail in certain well-known ways :—

(1) Failure of rivets in shear.
(2) Tearing of plate through a line of rivet-holes.
(3) Combined tearing of plate and shearing of rivets.
(4) Lap rupture.

These four possibilities form the basis for the design of riveted joints ; (1) is accounted for by providing an adequate section of rivet-material in shear, (2) by providing an adequate section of plate to resist the tearing process, (3) by taking account of the weakest section of the plate and adding to its strength the resistance of the rivets which would have to shear in order to allow the tearing to take place, and (4) by providing adequate material between rows

of rivet-holes and between the holes and the plate-edges. These are the main factors which influence the design of riveted joints, but it is not suggested that other considerations are not of some importance. For example, the resistance of a riveted joint to forces tending to rupture might be considerably decreased due to severe or improper conditions of manufacture and service.

It is, however, not the purpose of this Paper to consider all the possible factors which might contribute to failure. Deterioration of material due to embrittlement and strain-hardening, crushing of rivet and plate due to excessive riveting pressure, cracking due to stress-concentration, fatigue and corrosion-fatigue are all important matters, but, whatever the primary cause, ultimate failure occurs in one of the four ways indicated above.

It will be appreciated, therefore, that it is not a difficult matter to assign a definite figure for the efficiency of a riveted joint. In general, the efficiency of a riveted joint may be defined as the ratio between the minimum calculated resistance to failure and the ultimate tensile strength of the solid plate, and this ratio is normally used in the calculation for working pressure. It should be noted that it is the ultimate static tensile strength of the plate material which provides the criterion of efficiency for riveted joints, and, so far as pressure-vessels are concerned, this criterion has been found by experience to be entirely satisfactory.

Consideration of the question of efficiency in welded joints appears at first to be a somewhat complex matter, mainly because of the various factors which influence the strength and soundness of welds to such an extent that joints of similar appearance and similar scantlings may be very dissimilar in physical properties. On the other hand, it may be thought that if the ultimate tensile strength of the plate is to be taken as the criterion of efficiency it should not be difficult to devise a suitable welding technique which will produce joints of pre-determined efficiency.

Welding development has at all times been directed towards the production of welded joints equal in every respect to the solid plate; that is to say, with a true 100-per-cent. efficiency. Whether in certain cases this ideal has or has not been achieved is a question of considerable difficulty. The answer must clearly depend upon the basis adopted for assessing the efficiency of a welded joint. It is in the adoption of a suitable basis that much divergence of opinion and practice is evident, and it might be as well at this point to examine some of the essential factors governing the strength-characteristics of welded joints.

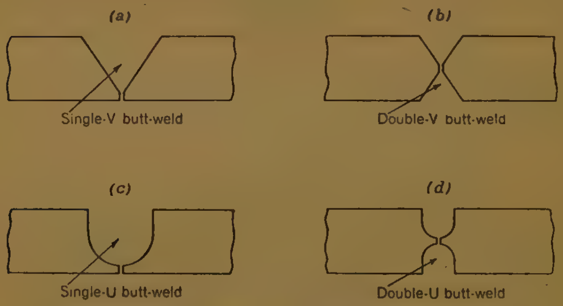
Design.

The introduction of welding for the construction of pressure-vessels does not relieve the designer of any responsibility. His work should not consist merely of fixing a suitable plate-thickness depending upon the allowable joint-efficiency. Sometimes, however, being ignorant of the details of the welding process involved, the designer is unable to give any indication of the size, shape, and position of the joints, and these important matters are in consequence left to the shop foremen, or even to the welders themselves.

There are four types of fusion butt-welded joints suitable for pressure-vessels :—

(*a*) Single-V butt-weld.
(*b*) Double-V butt-weld.
(*c*) Single-U butt-weld.
(*d*) Double-U butt-weld.

Figs. 1.



Types of Welds.

These types are shown in *Figs. 1* and, whilst there are a number of variations depending upon angle of V and on whether the plate edges are fully or only partially chamfered, the main point to note is that in all cases the weld is made from both sides of the plate in order to ensure full penetration of the weld-metal.

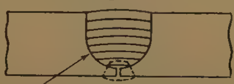
Before the precise dimensions of a joint can be determined it is necessary to take into account the shrinkage characteristics of the weld-metal which is to be deposited. This is largely a question of experience and it is usual in V butt-welds to allow a gap at the root of the weld which varies between $\frac{1}{16}$ and $\frac{5}{32}$ inch, depending upon the plate-thickness. In the case of boiler-drums manufactured in Great Britain, however, the U-type of joint has been found to be the most suitable. The welding procedure adopted necessitates cutting out from

the underside of the joint the whole of the first run of weld-metal deposited in the U. On account of this, and of the fact that only high-quality ductile weld-metal may be deposited, it is unusual to allow any definite shrinkage-gap whatever, and the plates forming the joint are butted as closely as practicable. This form of joint is shown in *Fig. 2*. It has a number of practical advantages including the following :—

(1) A uniform deposition of weld-metal in each layer is possible.
(2) Straight runs are possible with a minimum amount of weaving.
(3) It is specially suitable for automatic welding.
(4) It facilitates X-ray examination, because the sides of the joint are practically in line with the direction of the rays.

A further important point in regard to the design of welded joints is that of position. In any welded structure the aim should be to place the welded joints at positions free from stress-concentrations,

Fig. 2.



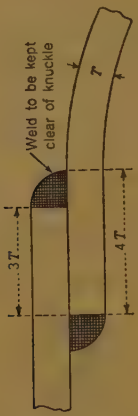
U BUTT-JOINT FOR PRESSURE-VESSELS.

and to ensure, so far as is possible, uniformity of the service stresses likely to be imposed on the weld. It is not always possible to achieve this ideal in the welding of complicated structures, but a cylindrical pressure-vessel offers ample scope for simplicity both in design and in welding practice. Where, however, joints have to be made between plates of unequal thickness, such as between shell- and tube-plates, or between shell-plate and dished end-plates, it is necessary to reduce the thickness of the thick plate to that of the thin plate in the vicinity of the weld. Where a vessel is of such length that it has to be made up in several sections joined together by girth-seams, the longitudinal welded joints should be staggered so that no two are in a direct line.

With a view to avoiding undesirable stress-concentrations in the welded joints, holes for boiler-mountings should be cut in the solid plate remote from the joints. It will be appreciated that this is a reasonable precaution having regard for the necessity of allowing the welded joints to develop their maximum efficiency. The question of allowable working stresses in the welded joints of pressure-vessels will be discussed later after consideration of other important factors involved in the assessment of joint-efficiency.

The lap-joint with inside and outside fillet-welds represents a form of joint which is sometimes used for the attachment of dished ends to cylindrical drums. Due to the force "couple" which exists when such a joint is strained in tension, it cannot be recommended for high-class pressure-vessels, and certainly should never be used

Fig. 3.



LAP-WELDED JOINT FOR DISHED ENDS.

for the longitudinal seams. In the design of a lap-joint for dishe ends there are two important factors :—

(1) There must be sufficient overlap in order to obtain reasonable flow of stress-lines.

(2) The outside fillet-weld must be clear of the knuckle of th dished-end flange.

Appropriate proportions for such a joint are indicated in *Fig. 3.*

WORKMANSHIP.

In calculating the joint-efficiency for a riveted joint, it is take for granted that the workmanship entailed in making the joint satisfactory. It is true that the relative strengths of the rive material and plate-material are taken into account, but the calcul tion assumes that the rivets are not subject to bending, and th they are properly driven so that each rivet will take its fair share the load.

Similarly, with welded joints the allowable efficiency or ratio joint-strength to the strength of the solid plate must depend up a reasonably good standard of workmanship. Whether the fact of workmanship is any more important in welding than in riveti

is a question open to argument, but there can be no doubt that the quality of a welded joint is more easily affected by human inconsistencies than is the quality of a riveted joint.

Nevertheless, experience indicates that there has been a tendency in the past to place too much emphasis on the "human element" in welding. The average standard of workmanship in a high-class welding shop which is properly supervised is without doubt good enough for the work generally undertaken, and whilst the quality of workmanship in hand welding in such a welding shop will vary to some extent throughout each run of weld-metal deposited, it is difficult to visualize the production of a welded joint which is wholly bad due to defective workmanship.

It will, however, be argued that any defect in a fusion-welded joint, whether isolated or not, and whether due either to bad workmanship or to any other cause, is a serious matter because of the possibility of the defect spreading throughout the full length of the joint. On the other hand, a faulty rivet is unlikely to transmit its defect to its neighbour, whilst a crack in the plate might reasonably be expected to confine itself to a portion of the plate between two adjacent rivet-holes.

Such arguments, however, should be qualified by consideration of the nature of defects which can be attributed to bad workmanship, and those that are due to other deficiencies in welding technique, which normally should be detected under an adequate system of procedure control.

Defects which might be due to faulty workmanship are as follows:—

(1) Imperfect fusion with parent-plate.
(2) Slag-inclusions.
(3) Gas-pockets.
(4) Contamination of weld-metal.
(5) Overheating of weld-metal or parent-plate.

Items (1), (2), and (3) and to some extent (4) will be detected by X-ray examination which must, in consequence, play an important part in the assessment of joint-efficiency. Item (4) indicates defects due to large fluctuations in arc-length, improper manipulation of electrodes, or, in the case of oxy-acetylene welding, an improper admixture of the gases in the flame. Under a careful system of supervision it is unlikely that these defects will be extensive throughout a welded seam, and the total effect of such local contamination (local not only in relation to the length of the seam but also in relation to the depth) need not be regarded with undue alarm. With regard to (5), serious overheating (that is to say, overheating

which cannot readily be rectified by subsequent heat-treatment) is unusual in the case of electric-arc welding as carried out in high-class welding shops, because the factors of current values and electrode fluidity should always be subject to special control. A possible cause of overheating, however, would be the maintenance of the arc in one position for too long a period. Fortunately, the defect would be readily revealed by visual inspection, due to the fact that it would be accompanied by serious undercutting at that particular position.

The purport of the foregoing remarks on workmanship is to show that this factor can without difficulty be accounted for in assessing the joint-efficiency, by providing

(1) adequate supervision of the welding work;
(2) careful inspection of each run of weld-metal;
(3) X-ray examination of the finished joint.

It is the practice with reputable firms in Great Britain for an experienced supervisor to be responsible for the detailed inspection of each run of weld-metal as deposited in the welded joints of important pressure-vessels. His duties also include the supervision of the "fit-up" of the vessel for welding, and the control of the current values. Such close supervision must be regarded as an essential factor in fusion-welding, as it is only by means of this supervision that the reliability of workmanship can be accepted as equivalent to that involved in high-class riveting for boiler construction.

Material.

It might be suggested that, apart from considerations of workmanship, a figure for the efficiency of a welded joint could be obtained by comparing the physical properties of weld-metal, as deposited in a joint, with the physical properties of the parent-plate. Unfortunately, the question of weldability of plate-material has no received the attention which such an important factor would appea to merit. The result is that there is some divergence of opinion i industry regarding the precise details of chemical analysis an physical properties which should be found in boiler-plate suitable fc welding.

The problem would perhaps be easier if the weld and the plat could be considered as separate and distinct components in th joint, in the same way as rivets and plate. It is not difficult t understand, however, that when molten weld-metal is run betwee the solid surfaces of two adjoining steel plates, these surfaces a themselves reduced to a molten condition and an intimate mixin

of the two fluids takes place. The fusion zone of a welded joint consists, therefore, of a diffusion of constituents of the parent-metal into the weld, and vice-versa. It is for this reason that special consideration should be given to the chemical and physical properties of steel plates intended for welding. In the circumstances it will be appreciated that a direct comparison between the properties of the mild-steel plate and the properties of all-weld-metal as deposited could be misleading in view of the fact that the weakest section of the joint might be in the fusion zone.

The question of weldability is at the moment the subject of research, both in Great Britain and abroad. For the purposes of this Paper, however, it will be sufficient to indicate the properties of plate material which has been successfully welded in the construction of boiler-drums, autoclaves, and other important pressure-vessels.

In Great Britain, the material usually specified for the construction of welded pressure-vessels is mild steel made by the acid or basic open-hearth process. The ultimate tensile strength of the plates lies between the limits of 26 and 30 tons per square inch, or between 28 and 32 tons per square inch, depending upon the tensile range specified. The minimum elongation required on a gauge-length of 8 inches is 23 per cent. for the 26–30-ton steel and 20 per cent. in the case of 28–32-ton steel. In certain cases the plates are normalized after rolling, and it is good practice to anneal (stress-relieve) the plates after bending them to the required shape.

The analysis of such plates will vary slightly, but in the main will comprise the following :—

Carbon	from 0·15 to 0·25 per cent.
Silicon	,, 0·05 to 0·20 per cent.
Manganese	,, 0·40 to 0·60 per cent.
Sulphur	0·03 per cent.
Phosphorus	0·05 per cent.

Material conforming to this analysis, using the upper limits for carbon and manganese and the lower limit for silicon, should yield the following test-results in the as-rolled condition :—

Limit of proportionality	11·5 tons per square inch.
Yield-point	15·5 tons per square inch.
Ultimate tensile strength	29 tons per square inch.
Elongation	35 per cent. on four diameters.
Brinell hardness number	128.
Endurance limit (direct stress) . . .	± 12·4 tons per square inch.

Lower-tensile steel plates would be accompanied by lower carbon- and manganese-contents, but the range of silicon indicated in the

above analysis permits the use of steel in either the fully "killed" or semi-"killed" condition.

It has long been the practice in riveted-boiler construction to use steel which may be said to be fully "killed," and which in consequence possesses a silicon-content of about 0·2 per cent. For the purposes of welding, however, opinion is turning in favour of low-silicon steel. It is not the Author's intention to discuss the metallurgical conditions governing weldability, especially as such conditions are not yet agreed upon among the metallurgists themselves. It is, however, evident that manufacturers of welded pressure-vessels prefer to work with low-silicon steel, and further research may have to be carried out, not only in regard to silicon-content, but also in regard to the relationship between silicon and manganese in steel, before any definite recommendation can be made.

The requirements of other authorities in regard to plate-material for welding are as follows:—

United States.—In the United States of America, the various regulations are based on the requirements of the Power Boiler Construction Code of the American Society of Mechanical Engineers. These requirements permit a range of steel for welding purposes, the details of which are given in Table I (pp. 632 and 633). The maximum allowable carbon-content for steel considered to be of weldable quality is 0·35 per cent.

Germany.—In Germany regulations are in force in respect of forge welding of boiler-drums, but in general, fusion-welding is only permitted in cases where the welded seam is covered by butt-straps. It is the practice, however, to give special permission by means of Ministerial Decrees, to certain firms to enable them to manufacture fusion-welded boiler-drums without straps. The special permission takes the form of a licence granted by the Prussian Ministry for Commerce and Industry, and is only issued after an extensive series of official tests have been carried out. The existing regulations are now being revised and a draft of new regulations has recently been submitted to the Ministry by the Committee appointed for that purpose. This draft requires that works which intend to weld new boilers or to repair existing boilers by welding must be approved.

The analysis of plate-material most commonly used in Germany for welding purposes is:—

Carbon	from 0·1 to 0·3 per cent.
Manganese	0·5 per cent. (max.).
Phosphorus	0·05 per cent. (max.).
Sulphur	0·05 per cent. (max.).
Silicon	up to 0·25 per cent.

Steel associated with this analysis is usually supplied in two ranges :—

(i) From 34 to 41 kilograms per square millimetre with an elongation of from 28 to 25 per cent. on 200 millimetres, and a reduction of area of from 80 to 70 per cent.

(ii) From 41 to 48 kilograms per square millimetre with an elongation of from 25 to 20 per cent. on 200 millimetres, and a reduction of area of from 70 to 60 per cent.

Until recently steel of 48 kilograms per square millimetre was considered to be the limit for good weldability.

Whilst the existing German regulations do not include any stipulations in regard to impact-testing of boiler-plate the German Steam Boiler Committee have given considerable thought to the subject, and it is of interest to note that a Charpy test value of 10 metre-kilograms per square centimetre is regarded as appropriate for plate-thicknesses up to 15 millimetres, and 8 metre-kilograms per square centimetre for plate-thicknesses above 15 millimetres. These values are quoted in connexion with plate-material of the 34—41 kilograms per square millimetre quality, but in view of the well-known lack of consistency in impact-test results on rolled-steel plates the figures should not be regarded as a final criterion.

Recent progress in Germany has, however, made it possible to adopt welding also for steels of higher tensile strength, and for slightly-alloyed steels. These steels include the above-mentioned qualities of mild steel with the addition of 0·5 per cent. of molybdenum, the steel having a tensile strength of from 35 to 50 kilograms per square millimetre ; ordinary mild steel containing 0·25 per cent. of molybdenum and 0·25 per cent. of copper, with a tensile strength of from 41 to 53 kilograms per square millimetre ; a special non-ageing carbon steel with a tensile strength of from 47 to 56 kilograms per square millimetre ; and this steel with the addition of 0·25 per cent. of molybdenum and 0·25 per cent. of copper. The above-mentioned steels are approved for electric-arc fusion-welding, while in the case of water-gas welding approval has been given for the use of a mild steel containing 0·25 per cent. of molybdenum and 0·25 per cent. of copper, with a tensile strength of from 44 to 56 kilograms per square millimetre, and for a steel having reduced ageing properties and a tensile strength of from 44 to 53 kilograms per square millimetre.

Switzerland.—In Switzerland the regulations governing the construction of welded boiler-drums are issued by the Swiss Association of Owners of Steam Boilers (Schweizerischer Verein von Dampfkessel-Besitzern). These regulations, promulgated in January, 1932,

TABLE I.—AMERICAN SOCIETY OF MECHANICAL ENGINEERS

Specification.	Quality.	Grade.	Chemical analysis. Carbon: per cent. For plate-thickness of ¾ inch or under.	Chemical analysis. Carbon: per cent. For plate thickness over ¾ inch.	Chemical analysis. Manganese: per cent.	Chemical analysis. Phosphorus: per cent. Acid.	Chemical analysis. Phosphorus: per cent. Basic.
S.1	Flange.	—	—	—	0·3 to 0·6	0·05 max.	0·04 max.
	Firebox.	—	0·25 max.	0·30 max.	0·3 to 0·6*	0·04 max.	0·035 max.
S.2	Flange.	Grade A.	0·15 max.	0·17 max.	0·35 to 0·60	0·06 max.	0·04 max.
	Firebox.	Grade A.	0·15 max.	0·17 max.	0·35 to 0·60	0·04 max.	0·035 max.
	Flange.	Grade B.	0·20 max.	0·22 max.	0·35 to 0·60	0·06 max.	0·04 max.
	Firebox.	Grade B.	0·20 max.	0·22 max.	0·35 to 0·60	0·04 max.	0·035 max.
S.4	—	Class 1.	0·35 max.	0·35 max.	0·40 to 0·70	0·05 max.	0·035 max.
	—	Class 2.	—	—	—	—	—
S.26	Flange.	—	0·32 max.	0·35 max.	0·90 max.	0·04 max.	
	Firebox.	—	0·32 max.	0·35 max.	0·90 max.	0·035 max.	
S.27	—	Grade A.	0·35 max.		0·50 to 0·90	0·04 max.	0·035 max.
	—	Grade B.					

* 0·3 to 0·5 for plate-thickness of ¾ inch or under.

Power Boiler Construction Code.

Sulphur: per cent.	Silicon: per cent.	Tensile properties. Yield-point: lbs. per square inch.	Ultimate tensile strength: lbs. per square inch.	Elongation on 8 inches: per cent.	Reduction of area: per cent.	Remarks.
0·05 max.	—	$\frac{\text{U.T.S.}}{2}$	55,000 to 65,000	$\frac{1,500,000}{\text{U.T.S.}}$	—	—
0·04 max.	—	,,	55,000 to 65,000	$\frac{1,550,000}{\text{U.T.S.}}$	—	—
0·05 max.	—	,,	45,000	$\frac{1,500,000}{\text{U.T.S.}}$	—	—
0·04 max.	—	,,	45,000	$\frac{1,650,000}{\text{U.T.S.}}$	—	—
0·05 max.	—	,,	50,000	$\frac{1,500,000}{\text{U.T.S.}}$	—	—
0·04 max.	—	,,	50,000	$\frac{1,650,000}{\text{U.T.S.}}$	—	—
0·05 max.	—	,,	60,000	26 *on 2 ins.*	42	—
—	—	—	—	—	—	Not permitted for welding.
0·05 max.	0·25 max.	$\frac{\text{U.T.S.}}{2}$	70,000 to 82,000	$\frac{1,600,000}{\text{U.T.S.}}$	—	—
0·04 max.	0·25 max.	,,	70,000 to 82,000	$\frac{1,600,000}{\text{U.T.S.}}$	—	—
0·04 max.	0·25 max.	,,	65,000 to 77,000	$\frac{1,750,000}{\text{U.T.S.}}$ *on 2 ins.*	—	—
		,,	70,000 to 82,000	$\frac{1,750,000}{\text{U.T.S.}}$ *on 2 ins.*	—	—

specify two qualities of mild-steel plate, and whilst both qualities may be used for welding, Quality I is recommended by the Association for that purpose.

Quality I.
Ultimate tensile strength : 35–44 kilograms per square millimetre.
Strength to be used in design : 38 kilograms per square millimetre.

Quality II.
Ultimate tensile strength : 41–50 kilograms per square millimetre.
Strength to be used in design : 42 kilograms per square millimetre.
Yield-point for both qualities, not less than 0·55 of the ultimate tensile strength.

Table II gives the figures for elongation corresponding to the ranges of tensile strength covered by the two qualities of plate-material.

TABLE II.

	Quality I.			Quality II.		
Ultimate tensile strength: kilograms per square millimetre	35–38	38–41	41–44	41–44	44–47	47–50
Elongation on 10d or 11·3$\sqrt{A}$: per cent. . .	30–28	28–26	26–24	26–24	24–23	23–22

NOTE.—A deviation not exceeding 10 per cent. is permissible in respect of these elongation figures; d denotes diameter of round specimen, and A denotes cross-sectional area of rectangular specimen.

No specified chemical analysis.

Sweden.—In Sweden, whilst considerable development has taken place in regard to the welding of structures and unfired pressure-vessels, the welding of boiler-drums is not yet an accepted practice. No detailed regulations have been issued in respect of welded pressure-vessels, but in general it can be said that the plate-material used for water-gas welding has a minimum tensile strength of 38 kilograms per square millimetre and a carbon-content of from 0·12 to 0·15 per cent. The material is of the low-silicon (semi-" killed ") type. For fusion-welding, higher tensile strength is generally used, the material having a range of from 44 to 55 kilograms per square millimetre and a carbon-content of from 0·16 to 0·20 per cent.

Italy.—In Italy there are no detailed regulations except for a Ministerial Decree which deals broadly with the application of welding to pressure-vessels, and which merely states that the plates are to be of a quality suitable for welding.

It is, perhaps, remarkable that although evidence indicates that Continental engineers are fully abreast of welding developments, no authoritative attempt has been made to establish a construction code governing the details of welding as applied to pressure-vessels.

Even to-day the only authority to issue such a code in Europe is Lloyd's Register of Shipping, which, in July, 1934, published "Tentative Requirements for Fusion Welded Pressure Vessels intended for Land Purposes." It is anticipated, however, that the new German regulations already in draft form will be published in the near future.

Certain European authorities contemplate the acceptance of welded pressure-vessels in their official regulations in that they do not prohibit them. Others, such as the Swiss Association of Owners of Steam Boilers, go so far as to legislate for the design of welded pressure-vessels, leaving the details of construction and testing to the discretion of the expert inspectors. In other cases, for example in Czechoslovakia, construction is generally carried out in accordance with the requirements of the Boiler Construction Code of the American Society of Mechanical Engineers.

In France also, specifications are at present in course of preparation, and the material considered suitable[1] conforms to the following analysis :—

Carbon	from 0·05 to 0·15 per cent.
Manganese	from 0·3 to 0·6 per cent.
Silicon	trace.
Sulphur	less than 0·03 per cent.
Phosphorus	less than 0·03 per cent.
Ultimate tensile strength .	38 kilograms per square millimetre (± 3).
Minimum elongation . . .	22 per cent. on 200 millimetres.

Before concluding this survey of the requirements for plate-material suitable for welding, reference must be made to the recommendations contained in the Boiler Code issued by the Standards Association of Australia. The steel recommended is that conforming to the requirements of either Specification No. B.14 or B.15, as shown in Table III (p. 636).

The foregoing résumé of requirements for weldable steel plates serves to indicate the present practice in various parts of the world. It is well known, however, that certain analyses which come within the ranges specified are less suitable for welding by certain processes than others. For forge-welding, for example, a very low silicon-content has been found to be the best for weldability.

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[1] M. R. Meslier, "Construction and Survey of Welded Vessels intended for Compressed, Liquefied or Dissolved Gas." XI^{e} Congrès de l'Acétylène et de la Soudure Autogène. Rome, June, 1934.

TABLE III.

Specification.	Quality.	Method of manufacture.	Chemical analysis.		Ultimate tensile strength: tons per square inch.	Elongation on 4 diameters. Plate-thickness:		Yield-point: tons per square inch.
			Sulphur: per cent.	Phosphorus: per cent.		0·375 inch and over: per cent.	Under 0·375 inch: per cent.	
B.14	"A" (shell).	Open hearth.	0·05 max.	0·05 max.	28–32	20	17	$\frac{\text{U.T.S.}}{2}$ (minimum)
	"B" (flanging).	Open hearth.	0·04 max.	0·04 max.	26–30	23	20	$\frac{\text{U.T.S.}}{2}$ (minimum)
B.15	"A" (shell).	Open hearth or Electric.	0·05 max.	0·04 max.	28–32	20 on $8d$ 24 on $4d$		$\frac{\text{U.T.S.}}{2}$ (minimum)
	"B" (flanging).		0·04 max.	0·04 max.	26–30	23 on $8d$ 28 on $4d$		$\frac{\text{U.T.S.}}{2}$ (minimum)

In the light of his experience, and pending a definite lead from those at present engaged in research work on this important subject, the Author considers that, for welded steel pressure-vessels, material conforming to the following specification will be found entirely suitable:—

Chemical Analysis.

Carbon	from 0·15 to 0·25 per cent.
Manganese	,, 0·4 to 0·5 per cent.
Silicon	,, 0·05 to 0·1 per cent.
Sulphur	0·03 per cent. (max.).
Phosphorus	0·04 per cent. (max.).

Physical Properties (as rolled).

Ultimate tensile strength . . .	from 26 to 30 tons per square inch.
Yield-point	,, 15 to 19 tons per square inch.
Elongation on 4 diameters . . .	35 per cent.
Reduction of area	60 per cent.
Fatigue endurance	± 12·5 tons per square inch (direct stress).
Allowable working stress for temperatures not exceeding 650° F.	from 5·8 to 6·7 tons per square inch.

It may be mentioned that the mechanical properties of welds at high temperatures have been investigated by the Welding Research Committee of the Institution of Mechanical Engineers, and it has been shown that weld-metal as normally used in the construction of boiler-drums is not very different from the boiler-steels themselves, and that up to temperatures of saturated steam at which boilers are generally working, creep is very small at the usual working stresses. The investigation also showed that it is only in parts subjected to superheated steam that creep is to be seriously feared and that at temperatures above 300° C. the phenomenon of creep may become of importance. In this connexion it is suggested that at 490° C. the stress should be less than 4 tons per square inch.[1]

Welded Joints.

With the introduction of welded joints in pressure-vessels it was deemed desirable by certain authorities to apply more exacting methods of testing and inspection than were ever contemplated in the case of riveted vessels. This has been done to such an extent in Great Britain that the accepted standards of to-day may be open to some criticism from the point of view of economic and efficient commercial production. In many cases the view which appears to

[1] Second Report of Welding Research Committee. Proc. Inst.Mech.E., vol. 133 (1936), p. 5.

be taken is that unless a welded joint can be produced which is in every way 100 per cent. efficient compared with the parent-plate then the riveted joint is to be preferred. Such an attitude ignore the undoubted advantages of welded joints and confers an entirely unjustifiable favour on the riveted joint which is not expected to have an efficiency of more than say, 85 per cent.

During the last 2 or 3 years a large number of important welded pressure-vessels have been made under survey, and numerous authoritative test-results have been obtained under the supervision of the Surveyors to Lloyd's Register of Shipping. These results indicate that under a proper system of procedure control, welded joints in pressure-vessels can be accepted with confidence. In many cases they provide a joint superior to that which can be obtained by riveting and are comparable only with solid-forged or solid-drawn vessels.

Typical average results of a large number of routine tests carried out for official purposes are tabulated in Tables IV and V. These serve to indicate the properties which can be obtained in welded joints made under a proper system of inspection. In general specimens of all-weld-metal taken from actual joints built up by multi-run welding reveal greater hardness than the rolled mild-steel boiler-plate material. The ratio between yield-point and ultimate tensile strength is higher, and the ultimate tensile strength itself is generally greater by 2 or 3 tons per square inch. The elongation however, is seldom as high as is usually obtained in the plate, but the reduction of area, an important indication of ductility, is approximately the same for both plate and weld-metal. It has generally been found that impact-values obtained for weld-metal are more consistently high than for rolled plate. In this connexion the results obtained for boiler-plate (given in Table V) suggest that in the annealed condition an Izod value of 27 foot-lbs. may be considered representative.

The test-results obtained from welded joints, that is to say, on specimens cut transverse to the joint (as distinct from specimens of all-weld-metal), are of especial interest, as they are a more direct indication of the strength of the joint. In the tensile specimen unless the shape of the specimen is such as to force failure into the weld, the ultimate fracture invariably occurs in the plate and not in the weld. Owing to the greater hardness of the weld-metal, reduction of area ("necking") takes place on each side of the weld, and a good proportion of the elongation is confined to the plate-material. It must, therefore, be recognized that a welded joint creates a "hard spot" in a structure and must at all times be regarded as constituting a lack of uniformity in the structure of the material forming

TABLE IV.

Type of weld.	Plate-material.							Joint.							All-weld-metal taken from joint.						Remarks.
	Yield-point : tons per square inch.	Ultimate tensile strength : tons per square inch.	Elongation : per cent.	Reduction of area : per cent.	Impact (Izod) : foot-lbs.	Hardness number (Brinell).	Fatigue limit : tons per square inch.	Yield-point : tons per square inch.	Ultimate tensile strength : tons per square inch.	Elongation : per cent.	Reduction of area : per cent.	Impact (Izod) : foot-lbs.	Hardness number (Brinell).	Fatigue limit : tons per square inch.	Yield-point : tons per square inch.	Ultimate tensile strength : tons per square inch.	Elongation : per cent.	Reduction of area : per cent.	Impact (Izod) : foot-lbs.	Hardness number (Brinell).	
Fusion.	—	26–30	23 on 8 ins.	—	—	116	—	19·7	28·9	37·5 on 2 ins.	40	66	137	± 12‡	24·4	29·5	27·5 on 2 ins.	61·7	58	143	—
,,	—	26–30	23 on 8 ins.	—	—	121	—	19	29·4	33·5 on 2 ins.	36·5	57	135	± 9‡	24·1	29·9	24·5 on 2 ins.	44·9	67	138	—
,,	—	26–30	23 on 8 ins.	—	—	—	—	20·5	28·9	25·5 on 2 ins.	23	65	—	—	23·2	31·4	36 on 2 ins.	65·5	55	—	—
,,	15·7	26·2	31 on 2 ins.	—	—	126	—	18·8	30	35·8 on 2 ins.	47·4	62	135	± 13‡	26·6	29·8	26 on 2 ins.	45	44·5	146	—
,,	—	31	34·2 on 2 ins.	—	6·5*	156	—	—	30·4	20 on 3 ins.	—	11·6*	166	± 13‡	—	—	—	—	10·2*	146	—
,,	15·75	28·6	67 on 1 in.	55	13·4*	69†	± 12·4	18·6	29·5	35 on 1 in.	49	14·5*	75†	± 10·8	—	—	—	—	12·3*	77†	Fatigue results average of 36 tests. Alternating bending. (10 × 10^6 cycles.)
,,	14·7	27·7	65 on 1 in.	53	16·4*	71†	± 13	18·5	29·7	35 on 1 in.	51	12·5*	78†	± 12	—	—	—	—	12·6*	78†	Fatigue results average of 36 tests. Alternating bending. (10 × 10^6 cycles.)
,,	14·7	27·7	65 on 1 in.	53	16·4*	71†	+ 15	—	—	—	—	—	—	+ 12·7	—	—	—	—	—	—	Average results, direct tensile fatigue tests from 1 ton to a maximum at 500 repetitions per minute (2 × 10^6 repetitions).
,,	—	29·2	27 on 8 ins.	—	—	137	—	14	31·5	—	—	60	150	—	24·1	32·5	30·4 on 2 ins.	60·3	52	148	—
,,	—	26–30	23 on 8 ins.	—	—	131	—	19·7	28·6	27 on 2 ins.	—	—	143	—	—	—	—	—	—	131	Not annealed.
,,	—	26–30	23 on 8 ins.	—	—	115	—	17·6	27·9	47 on 2 ins.	—	—	126	—	—	—	—	—	—	126	Annealed.
,,	—	28–32	20 on 8 ins.	—	—	—	—	—	29·8	28·2 on 2 ins.	—	—	—	—	22·5	29·5	36·2 on 2 ins.	49·8	55	—	Average of 16 sets of tests by firm new to this class of work.
,,	—	28–32	20 on 8 ins.	—	28·5	131	—	—	31·9	29·1 on 1½ in.	—	42	141	± 13·7‡	—	32·6	25 on 2 ins.	48·3	31	133	—
,,	—	28–32	20 on 8 ins.	—	54	132	—	—	30·8	—	—	68	148	—	27·1	31·4	31 on 2 ins.	61·8	65	148	—

* Mesnager results : metre-kilograms per square centimetre. † Rockwell number. ‡ Wöhler results.

TABLE V.

TABLE V.

Firm.	Type of weld.	Plate-material. Yield-point: tons per square inch.	Ultimate tensile strength: tons per square inch.	Elongation: per cent.	Reduction of area: per cent.	Impact (Izod): foot-lbs.	Hardness number (Brinell).	Fatigue limit: tons per square inch.	Joint. Yield-point: tons per square inch.	Ultimate tensile strength: tons per square inch.	Elongation: per cent.	Reduction of area: per cent.	Impact (Izod): foot-lbs.	Hardness number (Brinell).	Fatigue limit: tons per square inch.	All-weld-metal taken from joint. Yield-point: tons per square inch.	Ultimate tensile strength: tons per square inch.	Elongation: per cent.	Reduction of area: per cent.	Impact (Izod): foot-lbs.	Hardness number (Brinell).	Remarks.
M	Water-gas	19·4	27·4	32 on 8 ins.	—	—	—	—	19·4	26·8	22·6 on 8 ins.	—	—	—	—	—	—	—	—	—	—	Tests on specimens ac from longitudinal j
N	,,	19·4	27·4	32 on 8 ins.	—	—	—	—	18·9	26·1	15·9 on 8 ins.	—	—	—	—	—	—	—	—	—	—	Tests on specimens ac from circumferenti
O	,,	20	27·3	29·2 on 8 ins.	—	—	—	—	18·8	25·4	12·6 on 8 ins.	—	—	—	—	—	—	—	—	—	—	Tests on specimens ac from longitudinal j
P	,,	20	27·3	29·2 on 8 ins.	—	—	—	—	19·6	26·6	14·3 on 8 ins.	—	—	—	—	—	—	—	—	—	—	Tests on specimens ac from circumferenti
Q	,,	—	28·1	29 on 8 ins.	—	—	—	—	—	26·8	17 on 8 ins.	—	—	—	—	—	—	—	—	—	—	—
R	,,	16·3	27	24·9 on 8 ins.	55·3	—	—	—	16·9	26·2	16 on 8 ins.	30·2	—	—	—	—	—	—	—	—	—	Tested at atmosphe perature.
R	,,	10·5	25·4	26 on 8 ins.	56·7	—	—	—	10	25	17 on 8 ins.	36·2	—	—	—	—	—	—	—	—	—	Tested at 400° C.
S	Unwelded plate.	17·7	28·5	25 on 8 ins.	48·1	20·6	—	—	Tests taken in direction of rolling: 1-inch boiler-plate (unannealed).													Average resul 48 sets of te
S	,,	18·8	28·8	22·5 on 8 ins.	43·7	18·8	—	—	Tests taken across direction of rolling: 1-inch boiler-plate (unannealed).													
S	,,	17·2	27·8	27·4 on 8 ins.	52·3	28·9	—	—	Tests taken in direction of rolling: 1-inch boiler-plate (annealed).													
S	,,	17·1	28·1	25·3 on 8 ins.	49·9	24·9	—	—	Tests taken across direction of rolling: 1-inch boiler-plate (annealed).													
S	,,	15	26·9	24·4 on 8 ins.	45·6	21·9	—	—	Tests taken in direction of rolling: 2-inch boiler-plate (unannealed).													
S	,,	15·3	27·4	22·9 on 8 ins.	40·3	20·2	—	—	Tests taken across direction of rolling: 2-inch boiler-plate (unannealed).													
S	,,	17·8	27·4	25·7 on 8 ins.	48·5	26·6	—	—	Tests taken in direction of rolling: 2-inch boiler-plate (annealed).													
S	,,	17·8	27·6	24·9 on 8 ins.	44·7	24·9	—	—	Tests taken across direction of rolling: 2-inch boiler-plate (annealed).													

the vessel. That this lack of uniformity is not serious is shown by the results of the fatigue tests, in which the maximum fluctuation of stress was, in all cases, concentrated at the weld.

The characteristic type of failure obtained in the static tensile specimen is mainly due to the yield-point of the plate being lower than that of the weld. It is only after the yield-point of the material has been reached that the property of ductility is really exercised; with this in mind the figures show that the ductility of good-quality weld-metal is quite as good as that of the plate.

The test-results given in Tables IV and V were obtained from practical tests, carried out in actual practice and not in laboratories. It has not, therefore, been possible to include any figures purporting to represent the elastic limit, but it would appear both from Tables IV and V and from examination of the specimens, that the weld-metal in the joint provides a greater elastic range than the plate-material. Accordingly, if the yield-point were used as a basis in design, it would be found that the factor of safety is always greater in the weld than in the plate. The fatigue-resistance of a welded joint is greatly affected by shape of contour, and by such defects as inclusions, bad penetration and undercutting. The direct tensile fatigue-test results given in Table IV for firm "F" were carried out on unmachined specimens, and show that the fatigue limit for the welded joint was 85 per cent. of that of the solid plate.

It is to be noted that the draft of the proposed new German regulations requires pulsating fatigue-tests to be carried out by firms desiring special dispensation, such as a joint-efficiency of 90 per cent. These tests in which the specimen has a surface finish identical to that of the boiler-shell are required for initial approval of the firm, and not as routine tests.

The Author's experience in the testing of welded joints leads him to regard a figure of $\pm$ 11 tons per square inch as a good average value for the Wöhler fatigue endurance-limit of high-quality welded joints suitable for pressure-vessel construction. It is considered that this figure is between 85 and 90 per cent. of that obtainable for boiler-quality plate.

Special attention should be drawn to the typical test-results obtained for water-gas forge-welding. This process of welding has been much longer established than the fusion welding processes, and for many years has been exclusively accepted for the construction of steam pipes, air receivers, etc. On the Continent, much experience has been obtained in the application of this process to the manufacture of boiler-drums, but the more recent advances on the part of fusion-welding processes have tended to retard the further development of water-gas welding for the manufacture of important pressure-

vessels working under severe conditions of stress, temperature and corrosion. It has, however, yet to be proved that water-gas welding can produce as good test-results as are obtainable with fusion-welding.

The ratio between the tensile properties of the joint and those of the unwelded plate are worthy of attention. There is practically no difference in the yield-points, and this is to be expected because there is no added weld-metal in the joint. The ultimate tensile strength of the joint varies between 93 and 98 per cent. of that of the plate, whilst the elongation obtained on welded specimens cut from longitudinal joints varies between 43 and 70 per cent. of that of the unwelded plate. The test-results obtained from specimens cut from circumferential seams indicate that there may be some inherent difficulty in effecting satisfactory welds in these positions. This, it is understood, is especially the case when the diameter of the vessel exceeds 4 feet. Whatever the cause, the weakness is apparent, and the Author is unable at present to suggest that better results are obtainable. The circumferential joints referred to in Table V give elongation figures which are only 49 per cent. of those obtained from unwelded plates. Figures for reduction of area for water-gas welded joints are between 54 and 64 per cent. of those obtainable in unwelded plates.

In putting forward these figures, which tend to throw an unfavourable light on an old and well-established method of welding, it is necessary to point out that the test-results obtainable for water-gas welding have at least the merit of consistency. Procedure control, whilst important, is not nearly so delicate as in fusion-welding, and it would appear that water-gas forge-welding is immune from the many variable chemical, electrical, and metallurgical factors so vital in the production of good-quality fusion-welds. For these reasons, it is unlikely that water-gas welding will ever be entirely supplanted in industry, and in many cases it will always be a preferable process both from the practical and economic points of view.

Joint-Efficiencies.

In considering the question "What is a reasonable figure to assign for welded-joint efficiency?" it would be well to take note of the joint-efficiencies and factors of safety allowed by those authorities which have legislated for welded joints in pressure-vessels.

Great Britain.—The only published rules governing design of Class I fusion-welded pressure-vessels are Lloyd's Register's "Tentative Requirements for Fusion Welded Pressure Vessels intended for

Land Purposes." These are at present under revision, but in the meantime they may be regarded as the accepted standard for both land and marine work in Great Britain. The formula for the allowable working pressure in the shell is

$$\frac{25{\cdot}5 \times S \times (t - 2)}{D} = \text{working pressure in lbs. per square inch,}$$

where

S denotes the ultimate tensile strength of the plate in tons per square inch,
t ,, ,, plate-thickness in thirty-seconds of an inch,
D ,, ,, internal diameter of the shell in inches.

The constant (25·5) represents a joint-efficiency of 82 per cent. combined with a factor of safety of $4\frac{1}{2}$, and this is equivalent to 91 per cent. with a factor of safety of 5.

As the result of the experience obtained since the Tentative Requirements were first issued, it is now considered that a joint-efficiency higher than 82 per cent. could be allowed for Class I pressure-vessels, and especially for those vessels operating under reasonably static conditions of temperature and pressure. Under the revised Requirements, joint-efficiencies up to 90 per cent. are permissible.

United States of America.—The Power Boiler Construction Code of the American Society of Mechanical Engineers permits a joint-efficiency of 90 per cent. combined with a factor of safety of 5. The formula for working pressure in lbs. per square inch is

$$\frac{TS \times t \times E}{FS \times R}$$

where

TS denotes the ultimate tensile strength of the plate in lbs. per square inch.
t ,, ,, minimum thickness of the shell-plate in inches.
E ,, ,, joint-efficiency = 90 per cent.
FS ,, ,, factor of safety = 5.
R ,, ,, internal radius of the shell in inches. If the shell-thickness is greater than 10 per cent. of the radius, the outer radius is to be used for R.

For temperatures above 700° F., the working stresses given in Table VI (p. 642) are specified in this Code in place of $\frac{TS}{FS}$ in the above formula.

TABLE VI.

Maximum temperature: ° F.	Minimum specified tensile strength of plate-material: lbs. per square inch.				
	45,000	50,000	55,000	60,000	75,000
	Working stresses: lbs. per square inch.				
700	9,000	10,000	11,000	12,000	15,000
750	8,220	9,110	10,000	11,200	13,000
800	6,550	7,330	8,000	9,000	10,200
850	5,440	6,050	6,750	7,400	8,300
900	4,330	4,830	5,300	5,600	6,000
950	3,200	3,600	4,000	4,000	4,000

For Class 2 unfired pressure-vessels the A.S.M.E. code specifies a joint-efficiency of 80 per cent. to be used in the above formula. The same joint-efficiencies are permissible by the Rules and Regulations of the Bureau of Navigation and Steamboat Inspection, U.S. Department of Commerce, but in this case the allowable factor of safety is 4½ instead of 5.

The specification governing the welding of pressure-vessels, issued by the Bureau of Engineering of the U.S. Navy, states that the joint-efficiency of welded joints under the cognizance of this Bureau shall be taken as 80 per cent. of the strength of the parent-metal except where the quality of the weld can be and is fully explored by radiographic or exographic photography, or other method satisfactory to the Bureau, in which case a joint-efficiency of 90 per cent. will be acceptable.

The joint "Code for the Design, Construction, Inspection, and Repair of Unfired Pressure Vessels for Petroleum Liquids and Gases," issued by the American Petroleum Institute in conjunction with the American Society of Mechanical Engineers, gives the following formula for allowable working pressure in the shell :—

$$\text{Working pressure in lbs. per square inch} = \frac{2SE(t-c)}{D_m}$$

where

S denotes the maximum allowable working stress in lbs. per square inch corresponding to the operating temperature.

E ,, ,, joint-efficiency.

t ,, ,, shell-thickness in inches.

c denotes the corrosion-allowance in inches.
D_m „ „ mean diameter in inches before the corrosion-allowance is added.

The maximum joint-efficiencies allowed by this Code for various

TABLE VII.

Type of joint.	Limitations.	Joint-efficiency: per cent.
Double-welded butt-joint.	None.	80
Single-welded butt-joint with backing-up strip.	Joints not over 1¼ inch thick.	80
Single-welded butt-joint without backing-up strip.	Joints not over ⅝ inch thick.	70
Double full-fillet lap-joint.	Circumferential joints only, not over ⅝ inch thick.	65
Single full-fillet lap-joints with plug welds.	Circumferential joints only, not over ⅝ inch thick.	65
Single full-fillet lap-joints without plugs.	Attaching dished ends convex to pressure, not over ⅝ inch thick.	55

types of welded joints are given in Table VII. These figures may be modified, however, by multiplying by certain factors as follows:—

(1) *Construction factors.*
Where shell-plates are made of firebox-grade steel to certain specifications of the American Society for Testing Materials, factor . . . . 1·0
Where shell-plates are made of flange-grade steel to certain specifications of the A.S.T.M., factor . . . . . 0·97
For certain other qualities of steel, factor . . . . . . 0·92

(2) *X-ray factor.*
When all the main welded joints of a vessel are:
radiographed, factor . . . . 1·12
not radiographed, factor . . . . 1·0

(3) *Heat-treatment factor.*
When a joint has been stress-relieved, factor . . . . . 1·06
„ „ not been stress-relieved, factor . . . . 1·0

It will be noted that for a double-welded butt-joint, which would be used for a Class I pressure-vessel, the highest joint-efficiency allowed under this Code would be:—

$$80 \times 1{\cdot}0 \times 1{\cdot}12 \times 1{\cdot}06 = 95 \text{ per cent.}$$

Switzerland.—The regulations of the Swiss Association of Owners of Steam Boilers (Schweizerischer Verein von Dampfkessel-Besitzern) specify the following formula for joints in tension :—

$$S = \frac{D.\,P.\,X.}{200\,K.Z.} + 1$$

where

P denotes the working pressure in kilograms per square centimetre.
D ,, ,, internal diameter in millimetres.
S ,, ,, minimum shell-thickness in millimetres.
K ,, ,, design strength of the plate-material in kilograms per square millimetre.
X ,, ,, factor of safety.
Z ,, ,, joint-efficiency.

The following factors of safety and joint-efficiencies are specified :—

Coke-fire welding (only acceptable in special cases).

Factor of safety	4·5	
Joint-efficiency, annealed	55	per cent.
Joint-efficiency, not annealed	40	,,

Water-gas welding (must be annealed).

Factor of safety	4·5	
Joint-efficiency, manual welding	65	per cent.
Joint-efficiency, machine welding	70	,,
Joint-efficiency, in special circumstances	85	,,

Oxy-acetylene welding.

Factor of safety	4·5	
Joint-efficiency for joints welded from one side only (not annealed)	50	per cent.
Joint-efficiency for joints welded from one side only, and annealed with blow-pipe	55	,,
Joint-efficiency for joints welded from both sides (not annealed)	70	,,
Joint-efficiency for joints welded from both sides and annealed in furnace	80	,,

Metallic arc-welding.

Factor of safety	4·5	
Joint-efficiency for joints welded from one side only	50	per cent.
Joint-efficiency for joints welded from both sides	70	,,

Germany.—The present practice in Germany in respect of welded boiler-drums is to allow a joint-efficiency of 90 per cent. to certain firms by means of special Ministerial Decrees, as previously explained. From the draft of the new Regulations it appears that it is proposed to allow joint-efficiencies of 70 per cent. for all kinds of welding, but special approval, which includes a comprehensive series of tests a

previously stated, will be required before efficiencies up to 90 per cent. will be allowed. Further, as mentioned later, every boiler-drum credited with a joint-efficiency in excess of 90 per cent. must be normalized after the completion of welding.

The foregoing summary of authoritative requirements regarding joint-efficiencies at present applicable to welded pressure-vessels is relevant to the consideration of the question of joint-efficiency. It will be observed that the figures accepted by these authorities vary between 40 and 95 per cent., depending upon such factors as welding process, heat-treatment, type of joint, and X-ray examination. These factors should be properly covered in any specification for the welding of important pressure-vessels, and it should be obvious that the question of joint-efficiency cannot be considered independently of these factors. Without adequate procedure-control, both in regard to details of construction and to system of testing, any assignment of joint-efficiency would be entirely unjustifiable. Accordingly, allowable joint-efficiencies must be correlated with conditions of manufacture, and graded according to the class of work involved.

The Author has shown that in certain aspects a welded joint may be over 100 per cent. efficient, whilst in other aspects the figure might be much lower. For example, the static tensile strength of a fusion-welded joint is invariably greater than that of the parent-metal, whilst the fatigue endurance may only be 85 per cent. of that of the solid plate. The requirements of other authorities are all based on testing experience, and in no case is there any suggestion that a joint-efficiency of 100 per cent. should be allowed. Quite apart from the consideration of test-results, however, there must always be an allowance for the variable uncertainties which are to be found even in the best type of joint, and after a careful study of the whole question, in the light of his own experience and the experience of other authorities, the Author is of opinion that for the highest class of fusion-welding, applied to pressure-vessels, and involving procedure control and comprehensive routine testing, the allowable joint-efficiency should not be greater than 90 per cent. The question of the factor of safety to be used in conjunction with the joint-efficiency is not within the scope of this Paper, but this will clearly be associated with the service conditions under which the vessels have to operate. In this connexion, fired pressure-vessels and vessels which might have to withstand severe fluctuations in pressure and temperature should be designed with a higher factor of safety than vessels operating under more static working conditions. Alternatively, if the factor of safety is kept the same for all welded pressure-vessels, the working stresses in the case of the former vessels should

be restricted by the figure allowed for the joint-efficiency. For the highest class of fired pressure-vessels, it is suggested that a joint-efficiency of 85 per cent. could be allowed. For high-class water-gas welding applied to boiler-drums, a joint-efficiency of 80 per cent. may safely be assigned for machine-welded longitudinal joints which have been properly heat-treated.

Procedure Control and Testing of Welded Joints.

One of the conditions under which a high joint-efficiency may be allowed in the case of welded pressure-vessels is that the vessel should be made under proper procedure control and adequate testing and inspection.

These important factors are interdependent, because no practicable system of testing could be devised which alone would be sufficient to indicate the strength qualities of the joint. It is therefore necessary for any inspecting authority responsible for the acceptance of welded pressure-vessels to be satisfied regarding the detail conditions under which the vessels have been constructed.

Procedure control embraces the following details :—

(1) Bending, preparation, and heat-treatment of plates.
(2) Assembly of plates prior to welding, and methods of support.
(3) Details of welding technique :
  (*a*) Process : metallic arc-, oxy-acetylene, or water-gas forge-welding.
  (*b*) Method of deposition of weld-metal : manual or automatic.
  (*c*) Materials : electrodes and filler-rods.
  (*d*) Current-values, gas-mixtures, forging-temperatures.
  (*e*) Cleaning, removal of slag, etc.
(4) Supervision during welding.
(5) Heat-treatment.

Each of these points is primarily the concern of the manufacturer, but quite apart from the results which might be obtained on specified routine tests, no inspecting authority would be wise in accepting a vessel unless entirely satisfied regarding these essential details of procedure control. The system adopted by Lloyd's Register of Shipping is to draw up a list of firms which, after exhaustive inquiry and special preliminary testing, have satisfied the Society's Surveyors that their procedure control is adequate. Such firms are thereafter under regular supervision. On this basis full credit can be given when routine test-results are obtained on individual pressure-vessels which comply with specified standards.

In this respect it is to be noted that in the draft of the new German Boiler Regulations it is stated that not only have tests to be carried out to demonstrate that the firm is capable of making efficient welds consistently, but, in addition, the equipment of the works for testing materials must be suitable for the testing of the welds and is to be checked and calibrated; further, the welders must be trained in accordance with approved standards and must have experience in the carrying out of high-quality boiler welding.

Fortunately, sufficient experience has now been obtained which enables satisfactory standards to be laid down. There is still, however, a tendency in Great Britain to adopt testing and inspection methods more suitable for the laboratory than the workshop. This attitude will, no doubt, be modified and rationalized in the light of further experience. It must be admitted, however, that it is not always possible for an inspecting authority to pay sufficient attention to the question of procedure control. This is a handicap not only to industry but to those authorities themselves.

For the purpose of routine testing it is usual to prepare coupon-plates of sufficient size to provide the required test-specimens. These plates are welded at the same time as the vessel which they are intended to represent, and in the case of fusion-welding are attached at each end of the longitudinal seam.

The test specimens usually comprise the following :—

For fired pressure-vessels, boiler-drums, and other vessels of primary importance.

(1) Transverse tensile-specimen.
(2) All-weld-metal tensile-specimen.*
(3) Transverse bend-specimen.
(4) Impact-specimens.†
(5) Density-specimens.*
(6) Micro- and macro-specimens.*†

For unfired pressure-vessels, air receivers, and vessels of less potential danger than those included in the above.

(1) Transverse tensile-specimens.
(2) Transverse bend-specimens.
(3) Transverse nicked bend-specimens.

The transverse tensile-specimen should be of the reduced-section type taken across the joint, and the ultimate static tensile strength

* In the draft of the new German Regulations Nos. (2) and (5) are omitted, and also macro-specimens.

† Nos. (4) and (6) are not included in the requirements of the Power Boiler Construction Code of the American Society of Mechanical Engineers.

obtained should be not less than the minimum strength of the plate. The thickness of the specimen should be equal to the thickness of the plate, and the contour of the weld should be the same as in the vessel. That is to say, it should not be dressed unless the actual seam is similarly treated.

It will probably be argued that this test-requirement at once assumes 100 per cent. joint-efficiency, and that if the actual designed efficiency required is only, say, 60 per cent., then a lower ultimate tensile strength should be acceptable. This would be true if the static tensile test was the ultimate criterion of strength, and if other properties such as ductility, toughness, and resistance to fatigue were consistently and uniformly related to the static ultimate tensile strength. It has, however, already been shown that joint-efficiency cannot be the simple ratio of $\frac{\text{ultimate tensile strength of joint}}{\text{ultimate tensile strength of plate}}$. In the circumstances, this requirement is not unreasonable, and, except perhaps for forge-welding, is not difficult to obtain.

The all-weld-metal tensile-specimen is usually taken in the case of fusion-welded joints. The specimen is cut in a longitudinal direction from the joint in the coupon-plate and, as the name implies, consists entirely of weld-metal. The specimen is of round section and its dimensions should conform to the accepted standards depending upon the thickness of the coupon-plate from which it is trepanned. This test is important as it indicates the tensile properties of the weld-metal actually deposited in the joint. For this reason the position from which the specimen is cut in the coupon-plate should be carefully selected with the aid, when possible, of X-ray examination. Whilst the weld-metal in the specimen should be representative of that in the actual joint, it would, however, be unfair if the results obtained were unduly influenced by isolated gas-holes and slag-inclusions. Here, again, the ultimate static tensile strength obtained should be not less than the minimum strength of the parent-plate. This should be accompanied by satisfactory figures for elongation on a given gauge-length and reduction of area.

The following figures applied in a recent case to the fracture of an all-weld metal tensile-specimen in connexion with the testing of a boiler-drum :—

Ultimate tensile strength	31·4 tons per square inch.
Yield-point	23·2 tons per square inch.
Elongation on 2 inches	36 per cent.
Reduction of area	65·5 per cent.

These figures are much better than could normally be specified

in a code of requirements, and whilst they serve to indicate what can be obtained in high-class welding to-day, experience has shown that the following figures provide a suitable standard :—

Elongation on 2-inch gauge-length	20 per cent.
Reduction of area	30 per cent.

Much has been written in technical literature concerning bend-tests on butt-welded joints, and there is no doubt that this simple and long-established test lends itself to extremely complex mathematical analysis. In any system of routine testing it is essential, if it is to be successfully applied in industry, that the interpretation of the results obtained on individual test-specimens should not involve abstruse calculation or complicated academic deduction. In other words, the test must be simple and the inference to be drawn from the result must be straightforward.

In the circumstances it is the Author's opinion that no highly scientific significance should be attached to a routine bend-test, and whether the result of this test is measured by angle of bend or percentage-elongation on the outer fibres is immaterial to the fact that the bend test itself is simply an obvious demonstration of the capacity of the material to withstand deformation without fracture. The test is therefore a comparative one, and for this reason all specimens should conform to certain standard dimensions.

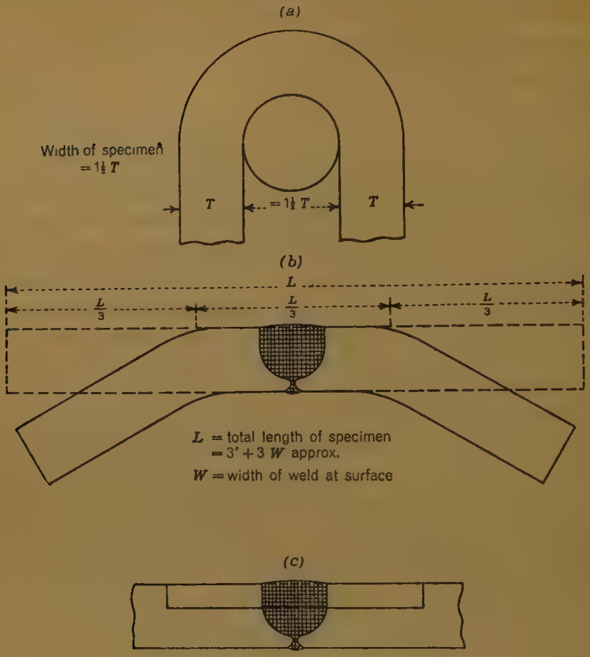
It is true that the degree of severity involved in testing a bend-specimen is governed by such factors as the shape and size of the specimen and the method of loading. Investigators have shown how these factors may be varied in order to obtain the best or worst results, as the case may be, but so long as a standard bend-test is adhered to, these questions are of limited interest.

Three forms of bend-test are in general use. These are shown in *Figs. 4* (p. 650).

(*a*) *The former bend-test*, in which the specimen, of width $1\frac{1}{2}$ times the thickness, is bent around a mandrel the diameter of which is $1\frac{1}{2}$ times the thickness of the specimen. A variation of this test is that prescribed in British Standard Specification No. 709 (1936) for testing weakness at junctions and soundness of weld-metal. In this case, the distance between the rollers supporting the bend-specimen is equal to the diameter of the former or mandrel plus 2·2 times the thickness of the specimen.

(*b*) *The free bend-test*, as prescribed in the Power Boiler Construction Code of the American Society of Mechanical Engineers. The specimen, of width $1\frac{1}{2}$ times the thickness, has an initial bend at each end and is placed as a

Figs. 4.



Types of Bend-Specimens.

strut in the compression machine; pressure is applied gradually at the ends until failure occurs in the outer fibres of the specimen. This test is also prescribed in British Standard Specification No. 709 (1936) for determining the ductility of weld-metal. A modified form of this test described in this Specification, but not in general use, involves the bending of a specimen at right angles to the weld.

(*c*) A small bend-specimen of rectangular section ¾ inch wide by ⅜ inch thick is machined from the upper surface of the coupon-plate transverse to the joint. The specimen thus includes weld-metal from the outer layers which is least affected by subsequent runs. The specimen is bent under free bending conditions until the arms are parallel, and the distance between the arms is not greater than ⅜ inch.

With bend-specimens (*a*) and (*b*), the results are judged either by the angle of bend obtained at fracture, or alternatively, the per

centage-elongation on the outer fibres measured on a gauge-length which extends across the width of the weld.

The following results should be obtained in the case of high-quality welding :—

Specimen (*a*)	180-degree bend.
Specimen (*b*)	30 per cent. elongation (minimum).
Specimen (*c*)	as stated.

It is to be noted that in the draft of the new German regulations the bend-specimens must withstand being bent through 180 degrees without fracture around a pin equal to

single thickness for plates of tensile strength of from 35 to 44 kilograms per square millimetre ;
double thickness for plates of tensile strength of from 41 to 53 kilograms per square millimetre.
treble thickness for plates of tensile strength of from 47 to 56 kilograms per square millimetre.

The impact test is not usually applied for the purpose of testing boiler-plate material, but it has consistently been included in specifications for the testing of joints in welded pressure-vessels manufactured in Great Britain. In America neither the A.S.M.E. Code nor the Regulations of the Bureau of Navigation and Steamboat Inspection contain any reference to impact-testing, although it is understood that the question was carefully debated before these Rules were issued. The welding requirements of the American Naval Bureau of Engineering do, however, specify impact-testing as one of the routine tests for the welded joints of Class A.1 pressure-vessels, which includes fired pressure-vessels such as steam-, water-, and superheater-drums for boilers.

It should also be noted that the impact-test is included in the draft of the new German regulations as a routine test. The minimum impact-figures vary with the qualities of plate used and are :—

Tensile strength : kilograms per square millimetre.	Impact-values : metre-kilograms per square centimetre.	
	Test-piece 10 × 10 millimetres.	Test-piece 30 × 15 millimetres.
35 — 50	8	12
44 — 53	6	10
47 — 56	5	8

The test-piece 10 × 10 millimetres (10 × 7 at bottom of notch) is used for plates less than 12 millimetres thick, and the test-piece 30 × 15 millimetres (15 × 15 at bottom of notch) for plates less than 30 millimetres thick. Charpy tests are employed, and special attention is paid to the results of the test-pieces having the notch formed in the fusion zone.

Opinion regarding the necessity and importance of these tests varies, mainly because there is no well-defined relationship between impact-values and the physical properties indicated by other methods of testing. A special point of criticism in regard to the impact-test is the inconsistency of the results obtained, mainly, it is thought, on account of the sensitivity of small impact-specimens to atmospheric conditions, shape of notch, minute inclusions, etc. It is always advisable when making impact-tests to examine the notch carefully both before and after testing. The dimensions of the notch and the manner in which it has been machined have a very decided influence on the test-results. Further, if flaws are present in the section under test, then a false result, or at any rate a result which is not truly representative, might be obtained. It may be as well to mention at this point that, while the Izod test is more selective, it is probable that the Charpy specimen will give more consistent results upon which more reliable information can be based.

Provided the results of impact-tests are interpreted with caution (that is to say, that due account is taken of the various extraneous factors which may influence the results obtained), the test has some value in indicating a condition of crystalline structure favourable to the propagation of a crack. This may be considered a test of "toughness" or a test of "brittleness," yet both these properties may in certain circumstances be revealed in the tensile- and bend tests. It would appear, therefore, that the impact-test must be associated in some way with grain-size and it may be possible to estimate impact-values from micro-examination of the crysta structure in the material.

There is no doubt that the impact-test is extremely sensitive to th results of heat-treatment, and in consequence it is useful in detectin improper treatment. Again, it is possibly the most useful test of a for detecting ageing in steel or weld-metal. In the latter cas however, it is necessary to extend the test over a certain period whic renders it impracticable as a routine test.

The fact that the test cannot be directly correlated with oth types of test should not necessarily be considered as detracting fro its value. It is probably more reasonable to regard the impact-te as a check on certain physical properties which cannot be reveale by any other form of test except, perhaps, a fatigue-test. In ar

case, there can be little doubt that it is a test which is of considerable value in checking the structure of the material; in this connexion the correlation of impact-values with grain-size was recently indicated in a Paper by Dr. T. Swinden and Mr. G. R. Bolsover.[1]

The inclusion of the impact-test in any specification for welded pressure-vessels is a matter for very careful consideration. Experience has shown that it is a test which should be applied judiciously, and the Author suggests that it should always be called for when there is some doubt regarding heat-treatment or when there appears from micro-examination to be some abnormality in crystal-structure.

So far as welded joints are concerned it has been advocated that in making an Izod test the aim of the test should, as far as possible, be to test the toughness of the metal at the surface of the weld in the unrefined layers where a crack is most likely to commence. This entails careful selection of the position of the bottom of the notch.[2] From a theoretical point of view it is safe to say that a low Izod figure indicates that an incipient crack will easily and rapidly extend throughout the full section of the material, in which case final failure would probably occur before the crack could be detected. It does not follow, however, that equally disastrous failures may not occur even when high impact-values have been obtained, as crack-detection in welded joints is a matter of considerable difficulty.

If an impact-test is applied in practical routine testing, then, in fairness to the manufacturer and the inspecting authority alike, a sufficient number of specimens should be tested in order to obtain reasonably representative results. It is impracticable to concentrate what might be termed "laboratory investigation" on one or two small sections of the welded joint included in an impact specimen, and whilst it is true that the strength of a chain is dependent upon its weakest link, it has yet to be proved that the full strength of a joint in a welded pressure-vessel is unduly affected by very small areas of local metallurgical imperfections. Such imperfections should properly be accounted for in the allowable joint-efficiency used in design.

The density test has always been associated with fusion-welding. In the early days of welding, when porosity and slag-inclusions were more prevalent than they are to-day, and when industrial radiographic technique was still very uncertain, there was every justifica-

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[1] "Controlled Grain Size in Steel." Journal Iron & Steel Inst., Vol. 2 (1936), p. 463.

[2] An interesting discussion on the value and significance of impact testing will be found in: L. W. Schuster, "The Notched Bar Test applied to Steel and Weld Metal, with Special Reference to the Izod Test." British Engine, Boiler and Electrical Insurance Co., Ltd., Technical Report, 1935.

tion for determining the density of the deposited weld-metal. In a modern specification for high-class welded pressure-vessels, governing procedure control and necessitating a very high standard of inspection and testing (including X-ray examination of the entire length of each welded seam), it is doubtful whether the density test is of any special value. Suffice it to say that, in the testing of a large number of pressure-vessels constructed under survey during the last 2 years, the Author has not received a single report of failure in respect of density. The normal specific gravity of mild steel is about 7·83, and that of high quality homogeneous weld-metal varies between this figure and 7·8.

Micro- and macro-specimens are not usually required under American regulations, but micrographs taken from the weld, the zone of thermal disturbance and in the plate material are included in the draft of the new German regulations. The only published code which calls for this examination is that issued by Lloyd's Register of Shipping, "The Tentative Requirements for Fusion Welded Pressure Vessels intended for Land Purposes." In these requirements it is specified that photo-micrographs at 100 magnifications are to be taken representing the centre of the weld-metal in the joint, the fusion zone, the adjacent parent-plate and the plate remote from the weld. A photo-macrograph is to be taken across a section of the welded joint. All these photographs are to reveal a sound homogeneous weld having a normal refined and uniform grain-structure. Typical photographs are shown in *Figs. 5* and *6*.

As in the case of the impact-test, this requirement may be regarded as a check on heat-treatment. A photo-micrograph at 100 magnifications is, however, of definitely limited value, and it is doubtful whether an abnormal structure which could be detected at this magnification would not be directly indicated by the results of the mechanical tests. It is often maintained that the microscope should be used in the laboratory and not in the test-house, and there is no doubt a lot to be said against the application of laboratory methods to the routine testing of pressure-vessels. It should be noted that in accordance with the revised requirements for welded pressure-vessels, Lloyd's Register will not require micro- or density specimens for the purpose of routine tests.

The nicked bend-specimen provides a ready means for examining a fractured surface of the weld for penetration, gas-pockets, slag inclusions, and grain-structure. It is a practical workshop test usually specified in the case of welded pressure-vessels of secondary importance. The nicks are cut at the sides of the weld, so that when the specimen is gripped in a vice and struck by a sharp blow or blows it will fracture through the weld.

Figs. 5.

PHOTO-MICROGRAPH OF PLATE. × 100

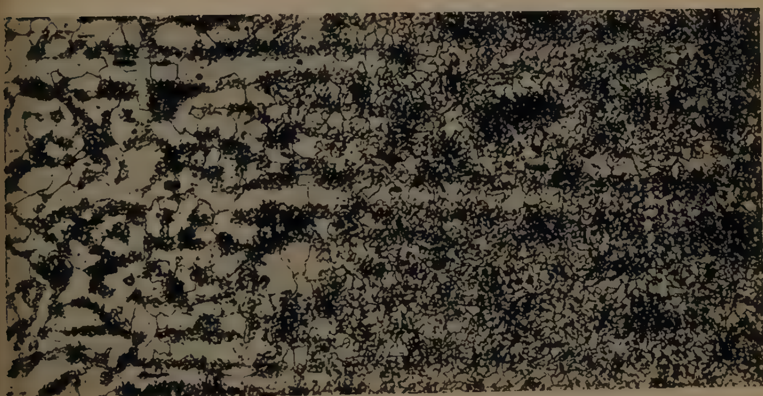

PHOTO-MICROGRAPH OF FUSION ZONE. × 100

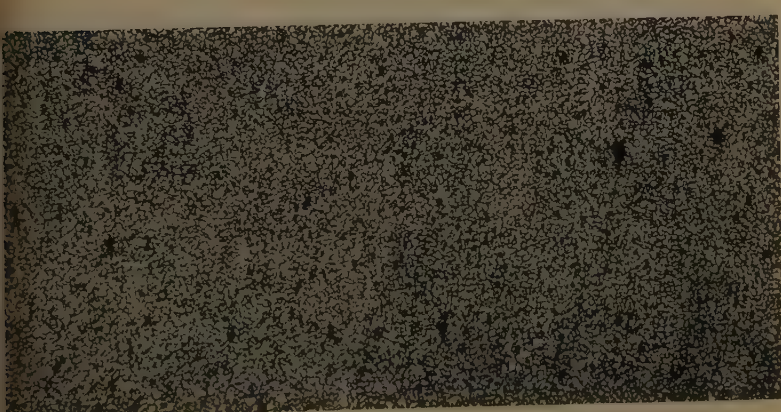

PHOTO-MICROGRAPH OF WELD-METAL. × 100

Figs. 6.

PHOTO-MICROGRAPH OF WATER-GAS WELD. ×

PHOTO-MACROGRAPH OF FUSION-WELD.

PHOTO-MACROGRAPH OF FUSION-WELD.

PHOTO-MACROGRAPH OF WATER-GAS WELDS.

In addition to the foregoing tests it is necessary for each vessel to be tested hydraulically to the appropriate pressure, and whilst under this test the vessel is to be well hammered on both sides of, and close to, the welded seams.

The foregoing remarks comprise a brief survey of the factors involved in procedure-control and methods of testing welded joints in pressure-vessels. The survey would not be complete, however, without reference to the two important subjects of heat-treatment and X-ray examination.

Heat-Treatment.

The Power Boiler Construction Code of the American Society of Mechanical Engineers, and other American specifications based thereon, are precise in their requirements regarding heat-treatment. It is specified that all fusion-welded drums or shells of power boilers shall be stress-relieved. This is to be done by heating uniformly to at least 590° C., and to 650° C., or higher if this can be done without distortion. The structure is to be held at the specified temperature for a period of time proportioned on the basis of at least 1 hour per inch of thickness, and is subsequently to be cooled slowly in a still atmosphere.

The draft of the new German regulations contains the requirement that welded parts are to be properly normalized after completion of the last welded joint, and a works' certificate to this effect is required. This would appear to be current German practice for high-class work, although there is some confusion of terms in the present regulations. The practice with a 35—44 kilograms per square millimetre mild steel is to heat to about 920° C. and to maintain at this temperature for about 1 minute per millimetre of wall thickness, with a minimum of 20 minutes.

There is no doubt whatever, both from the point of view of stress-relieving and from that of improving the physical properties of the weld-metal in the joint, that heat-treatment is desirable. The only question on which there may be some divergence of opinion is in regard to the temperature at which the treatment should be carried out. In the case of mild-steel vessels there are, in the main, two treatments available :—

(1) Stress-relieving at a temperature of about 600° C.
(2) Full normalizing at a temperature of about 930° C.

In both cases the vessel is usually removed from the furnace after a period of "soaking" and allowed to cool in a "still" atmosphere, which means that the vessel is placed alongside the furnace and

shielded from draughts by suitable means. The choice between these two treatments is influenced by the structural rigidity of the vessel at the temperature involved.

At temperatures above 900° C. there is a possibility of collapse of the shell of the vessel if it is not carefully supported, and whilst there is general agreement that heat-treatment at this temperature will produce the best results, manufacturers invariably prefer the lower-temperature treatment in order to avoid distortion troubles. That distortion in the case of thick drums can be avoided is evident from the fact that certain experienced manufacturers normalize their products as regular routine practice. The rigidity of a cylindrical vessel is largely dependent on the ratio between diameter and wall-thickness, and for the majority of welded boiler-drums this ratio is not too big to permit effective normalizing.

In this connexion it is of interest to note that American regulations usually exempt from any form of heat-treatment second-class welded pressure-vessels in which the ratio between diameter and wall thickness exceeds 100.

The question of cost of production also has an influence, and no doubt the above considerations have prompted the proposal contained in the draft of the new German regulations that, under certain conditions, normalizing may be dispensed with, and fusion-welded boilers may be accepted in the stress-relieved or even in the unannealed condition. In this connexion, and with reference to the special non-ageing steels used in Germany and mentioned in the section dealing with plate-material, it is of interest to note that the German authorities permit the omission of heat-treatment of welds made with special austenitic electrodes. Austenitic weld-metal may be regarded as a solid solution of carbon in iron, a condition which is stable through the effect of alloying elements. The weld-metal is therefore quite different from the parent-plate material.

It is claimed by the makers of this type of weld-metal that as this austenite does not harden through rapid cooling, heat-treatment is unnecessary. Further, it is pointed out that annealing at 600° C. does not effect any appreciable change in the crystal structure of the weld, whilst a heat-treatment of 900° C. brings about a definite breakdown of the austenite, causing an undesirable precipitation of carbide.

From the metallurgical point of view, there is much to be said in favour of the omission of heat-treatment in the case of austenitic welds; but to do so is to discount the significance of a narrow zone of martensitic structure which invariably occurs in the transition area between weld-metal and plate. The width of this zone has been found to vary up to a maximum of about 0·04 millimetre, and it is

authoritatively claimed that this so-called martensite occurring in low carbon-chrome mild steel is not a brittle element. In other words, it must not be confused with the "cutting hard" martensite of a hardened steel. Investigations have shown that whereas the Brinell hardness-number of the latter is about 475, that of the martensite in question is only about 260. It is also claimed that this narrow band of martensite had no deleterious effect on the results of mechanical tests.

Two comments might be made in criticism of the claims made for unannealed austenitic welds. Firstly, the Author has seen no evidence to prove that the extent of the martensitic zone can consistently be restricted within narrow limits for all thicknesses of plate, and in any case the presence of martensite, soft or hard, indicates a tendency to instability at the fusion zone. Secondly, the authoritative claims put forward in favour of the omission of heat-treatment do not take sufficient account of the question of residual stress and the necessity for its relief.

The subject is dealt with by K. L. Zeyen,[1] who discusses the results of mechanical tests made on austenitic welds in the unannealed, stress-relieved, and normalized condition. He concludes that, whilst such welds can be heat-treated, the principal advantage of austenitic welding lies in the fact that annealing or normalizing are not necessary.

The conditions governing the acceptance, under the German regulations, of welded boiler-drums annealed at from 600° to 650° C. in order to relieve, as far as possible, the heat-stresses due to welding are

(1) The allowable joint-efficiency shall not exceed 70 per cent.
(2) The faultless quality of the welded joint shall be demonstrated by non-destructive test after completion.
(3) The weld shall have sufficient ductility.

It should be noted that exemptions from normalizing are confined to fusion-welding. For example, every water-gas-welded boiler-drum has to be normalized because the overheated structure frequently found in portions of an unannealed water-gas weld cannot be rectified by low-temperature annealing.

It is sometimes necessary in the case of very long vessels to carry out the heat-treatment in sections, because available heat-treating furnaces are not large enough to accommodate the whole of the vessel at one time. This procedure is acceptable so long as sufficient overlap is allowed to ensure the uniform heating of the entire welded

[1] "The Question of Annealing Treatment in Austenitic Boiler Welding." *Technische Mitteilungen Krupp*, vol. 4 (1936), p. 162.

TABLE VIII.—IZOD IMPACT TESTS.

Condition.	Tests on weld-metal.							Tests on plate.			
	Test No.	Yield-point: tons per square inch.	Ultimate tensile strength: tons per square inch.	Percentage elongation on		Reduction of area: per cent.	Impact: foot-lbs.	Plate-surface		Trans-verse to plate: foot-lbs.	Transverse adjacent to weld: foot-lbs.
				$4\sqrt{A}$	8*d*.			with † rolling: foot-lbs.	across † rolling: foot-lbs.		
As welded	1	24·0	29·7	26·8	19·7	37·7	45, 53	20, 18	32, 47	32	33, 39
	2*	27·8	33·5	12·0	8·8	30·0	50, 27	18, 17	22, 19	20	10, 11
	3	26·7	29·0	23·3	16·6	34·0	56, 48	27, 35	42, 50	30	18, 22
Stress relieved at 550–600° C.	4	25·0	28·1	26·7	20·6	39·5	50, 54	18, 17	32, 47	28	24, 51
	5*	27·1	28·8	2·1	3·75	12·3	10, 40	19, 20	24, 34	37	30, 49
	6*	24·8	28·3	12·3	11·5	18·0	47, 56	30, 30	49, 49	30	37, 31
Normalized at 900° C.	7	22·4	27·2	33·3	25·6	43·4	53, 38	34, 36	66, 64	62	70, 77
	8*	20·8	27·4	13·4	12·5	24·0	27, 44	44, 41	68, 67	66	77, 76
	9	22·2	27·6	34·0	23·5	46·1	46, 52	41, 36	55, 58	40	53, 47

* Faulty specimens due to slag-inclusions or gas-pockets. † Direction refers to the notch of the Izod specimen.

seam. Heat-treatment by means of blow-lamps and gas-rings is of doubtful efficacy, especially that by means of blow-lamps, as there is no definite means of temperature-control. The stress-condition of a joint after blow-lamp treatment may be worse than if no heat-treatment had been applied.

In order to obtain some indication of the effect of heat-treatment on the physical properties of deposited weld-metal a series of tests were carried out at the instigation of the Author. The specimens were cut from three coupon-plates, one of which had received no heat-treatment, the second had been stress-relieved at 600° C., and the third had been fully normalized at 900° C.

The results obtained are given in Table VIII, and *Fig.* 7 (facing p. 660) shows the location of the Izod impact-specimen cut in the welded joint. It will be noted that the impact-values for the deposited weld-metal remained unaffected by the heat-treatments given, and, in all three conditions were satisfactory. These tests were carried out on flat plates. The tensile strength of the weld-metal in the "as welded" condition was reduced by 13 per cent. when stress-relieved, and by 18 per cent. when normalized at 900° C. The ductility, however, although unaffected by stress-relieving, was considerably improved when the material was normalized.

It may be stated, however, that owing to the complete removal of internal stresses in the weld by normalizing, higher impact-values are to be expected on test-pieces which have received this superior heat-treatment.

The general conclusion to be drawn from these tests is that, whilst the best condition as regards ductility is obtained when a vessel is normalized above the upper critical temperature, a satisfactory welded joint can be obtained by means of a stress-relieving heat-treatment only. It may be added that this practice is almost universally adopted by manufacturers of welded pressure-vessels and should continue to give satisfactory results. Available research-data indicate that approximately 90 per cent. of the residual stress in a welded joint can be relieved at a temperature of 600° C.

A possible method of relieving internal stress in welded joints is by means of an externally-applied static loading. If the residual stresses in or adjacent to the weld are in the neighbourhood of the respective yield-points of the deposited metal and parent-plate then it will require only a moderate superimposed load to cause plastic flow to take place at the points of high local stress, provided that the highly-stressed portions possess adequate ductility at the yield-point. There is experimental evidence to show that this action does occur and that the yielding and mechanical stress-relief should not be accompanied by a reduction in impact-value provided that the

material is sufficiently ductile. The Author has no information that this method has ever been applied in practice, but it is interesting to note that if the view is correct that residual stresses approximating to the yield-point are normally present in and adjacent to the weld, then the customary routine hydraulic test must afford a considerable measure of mechanical stress-relief.

Further information on this subject is given in a recent Paper by Dr. Lewis Reeve.[1]

X-Ray Examination.

One of the main arguments often made in the past against the welding of important pressure-vessels was the apparent impossibility of determining the actual condition and quality of the welded joints. With the development of industrial radiographic technique this argument is no longer tenable, and X-rays have now become the most valuable of the non-destructive tests applicable to pressure-vessels. In America and in Great Britain X-ray examination is regularly applied to Class I pressure-vessels, and is invaluable as a routine test.

It is, however, important to understand clearly the limitations of radiographic technique and to realize that it is possible for serious defects such as cracks to escape detection. The main difficulty involved in specifying X-ray examination of welded pressure-vessels is in the provision of a suitable standard which will differentiate between acceptable and non-acceptable welded joints. There are so many variable factors each of which affects in some way the resultant photograph that it is quite impossible to provide standard photographs without some measure of standardization in technique. It is in this respect that inspection-bodies are apt to be led astray, and it is not uncommon to find inspecting authorities in Great Britain attempting to judge X-ray negatives without any reference to the details of radiographic technique which play such an important part in their production. Such procedure is unlikely to engender confidence in inspection methods. Suitable standards can only be evolved as the result of experience, and for this purpose Lloyd's Register found it necessary to have the Surveyors concerned specially trained in the interpretation of X-ray negatives. Their experience in X-ray examination is gradually leading to a sufficient accumulation of knowledge and data to enable some measure of standardization to be effected.

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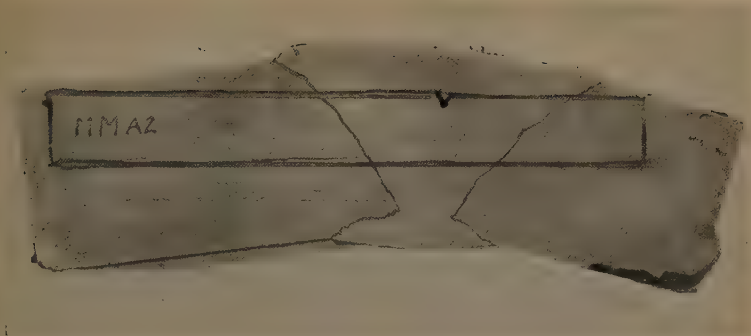
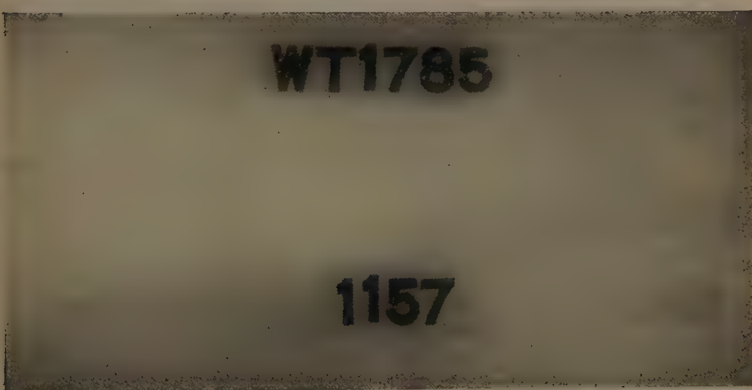
[1] "Internal Stresses in Welding and their Determination." *The Welding Industry*, vol. iv (1936), p. 344.

Fig. 7.



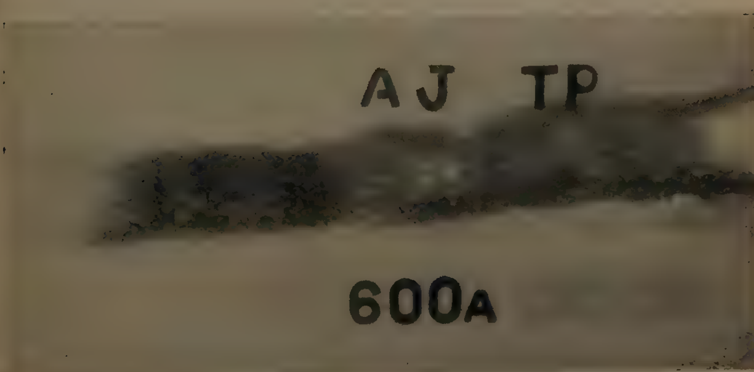
POSITION OF IZOD SPECIMEN IN WELDED JOINT.

Fig. 9.



A GOOD WELD: SURFACE GROUND.

Fig. 10.



UNACCEPTABLE POROSITY, BAD PENETRATION, CRACKS VISIBLE.

Fig. 11.

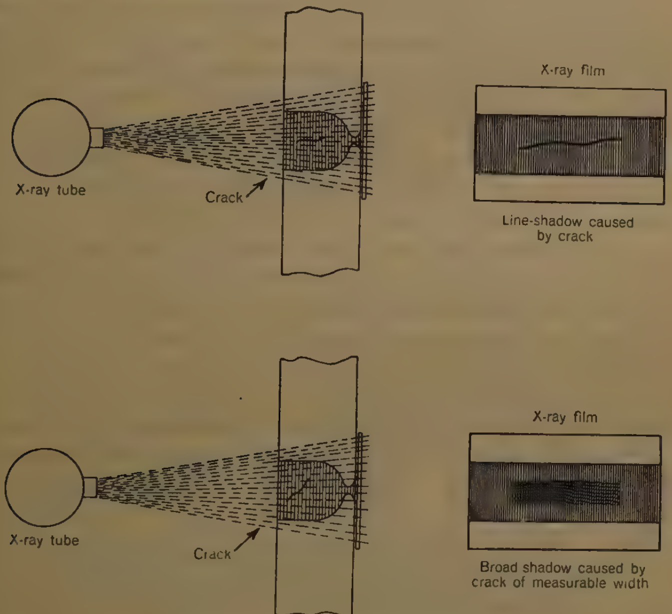
PROPANE STORAGE TANK: DIAMETER 10 FEET, LENGTH 40 FEET, WORKING PRESSURE 300 lbs. PER SQUARE INCH

X-ray examination is mainly directed to the detection of the following defects :—

(1) Porosity (slag- and gas-inclusions).
(2) Lack of penetration.
(3) Cracks.

In regard to (1) and (2) it is not difficult to set up satisfactory standards, but the detection of cracks, which are probably the most serious of weld-defects, entails a number of special problems.

Figs. 8.



X-RAY EXAMINATION OF CRACKS.

In the first place, if the depth of the crack is not in line with the direction of the rays, the negative may remain unaffected, especially if the width of the crack is less than $\frac{1}{1000}$ inch. If, however, the width of the crack is considerable, it will be revealed in the negative as a shadow, the dimensions of which will depend upon the angle between the crack and the direction of the rays (*Figs. 8*).

Obviously, in order to explore fully the possibilities of cracks in a welded joint X-ray photographs should be taken from a number of different angles so that if cracks are present line-shadows will be produced on the negatives. This is impracticable as a routine acceptance test, and the practice is to take photographs from one

direction only, namely, with X-rays normal to the surface of the joint, and to take further photographs from different angles when a crack is suspected. In this connexion it is not unusual to find cracks radiating in various directions from irregularly-shaped gas-pockets or slag-inclusions; consequently where such defects are indicated in the X-ray negatives, cracks may be suspected.

The only example of standardization of radiographic technique at present in practice is to be found in the Power Boiler Construction Code of the American Society of Mechanical Engineers. In this Code it is specified that the welds shall be radiographed with a technique which will determine quantitatively the size of defects with thicknesses equal to and greater than 2 per cent. of the thickness of the base-metal. A method for ensuring the fulfilment of this requirement is specified in detail and involves the use of thickness-gauges or "penetrameters" which are placed alongside the weld under examination so as to cause a variation in density in the X-ray negative. This variation corresponds to the thickness of the "penetrameter," which should not be greater than 2 per cent. of the metal thickness in way of the weld. Similar methods are employed in Great Britain, and Lloyd's Register's requirements specify that on each negative there is to be an indication of the relative shadow-density corresponding to 2 per cent. of the thickness of the weld.

In order to standardize technique, the A.S.M.E. Code also specifies the minimum distance from the source of radiation to the back of the weld, and in cases in which it is necessary to expose the film at a distance greater than 1 inch from the weld, a ratio of 7 to 1 is specified in respect of

$$\frac{\text{Distance from source of radiation to weld-surface toward radiation}}{\text{Distance from weld-surface toward radiation to film}}.$$

Rules are also laid down governing the system of identification of film with the portion of the welded joint which it is intended to represent.

Whilst these are all very important factors which lend themselves to standardization, there are other equally important factors which at present can only be regarded as variable:—

(1) The films.
(2) The fluorescent intensifying screens.
(3) The X-ray tube.

It will be appreciated that each of these is of fundamental importance in the production of X-ray photographs. Experience has shown that each can vary considerably, involving deliberate changes

in exposure time and distances, and the Author has found that the only satisfactory method of ensuring consistency is to take sample exposures at frequent intervals during the examination of a vessel.

Complaint is sometimes made regarding the cost of X-ray examination, and there is no doubt that this requirement adds considerably to the price of Class I welded pressure-vessels. The actual cost varies with a number of factors, such as length of each film and exposure-time, etc., but on an average it can be said that, in Great Britain, the minimum cost of each X-ray photograph of a length of 12 inches is about 5 shillings. This is quite apart from any consideration of cost of plant and depreciation, but includes labour charges and cost of materials both in the taking of the photographs and in their development. Additional charges may be incurred due to handling costs, depending upon such questions as factory lay-out.

Proposals are sometimes made that instead of subjecting the entire length of a welded seam to X-ray examination a series of photographs might be taken at certain selected positions, and it is suggested that these might indicate the quality of the joint as a whole. There is something to be said for this proposal in the case of certain pressure-vessels of less importance than, say, boiler-drums. At the same time, partial X-ray examination could on no account be regarded as thorough examination. At the most it would indicate the average quality of the joint in respect of porosity and inclusions. A complete X-ray examination involves the careful investigation of each inch of the seam for cracks, lack of fusion, and other defects previously mentioned. The value of such an examination depends entirely on the thoroughness with which it is carried out. Typical X-ray photographs are shown in *Figs. 9* and *10* (facing p. 660).

The use of magnetic or magnetographic methods for the examination of welded joints has made little headway, mainly because of the inability of such methods to reveal sub-surface defects. There is undoubtedly a scope for a non-destructive test other than X-rays, mainly in connexion with pressure-vessels which are not considered important enough to justify the expense of X-ray examination.

In many cases, a good practical test which, however, is partially destructive, is to trepan circular disks from the welded joint; these disks are then cut up as required and etched. Special tools are available for the purpose of trepanning such disks, but on the other hand there is no reason why they should not be cut by means of the oxyacetylene blowpipe. The holes must be filled afterwards by circular plugs welded in.

Conclusion.

In this Paper the Author has attempted to summarize the present position regarding the adoption of welded joints in pressure-vessels. It is thought that in conclusion it might be of special interest to note the number of fusion-welded Class I pressure-vessels which have been made in Great Britain and in America.

America.

Approximate number of fusion-welded boiler-drums (August, 1936)	Land	3,000
	Marine	770
Largest drum	144 inches diameter, working pressure 210 lbs. per square inch.	
Highest working pressure	1,400 lbs. per square inch. Diameter of drum	60 inches
Maximum plate-thickness used in construction of welded boiler-drums	$4\frac{15}{16}$ inches.	

In the construction of the Boulder dam, over 400,000 feet of welding was involved. This included 14,500 feet of Class I welding which had to be stress-relieved and X-rayed. The pipe-diameters varied between $8\frac{1}{2}$ feet and 30 feet, and the plate-thicknesses between $\frac{5}{8}$ inch and $2\frac{3}{4}$ inches.

In addition to the boiler-drums mentioned above, there have been 10,070 fusion-welded pressure-vessels made in accordance with the A.S.M.E. Code for Unfired Pressure Vessels, or the equivalent provisions of the A.P.I.–A.S.M.E. Code (American Petroleum Institute and American Society of Mechanical Engineers). Of these vessels, the largest was 20 feet in diameter, working at a pressure of 125 lbs. per square inch. The highest working pressure involved was 3,000 lbs. per square inch in a vessel of 36 inches diameter. The maximum plate thickness used in the construction of these vessels was 6 inches. It should be added that the Author has been informed by the Secretary of the American Boiler Manufacturers Association, Fair Practice Committee, that to their knowledge there has been no failure of any fusion-welded pressure-vessels made in America under the survey of the recognized authorities and in accordance with the requirements of the recognized Codes.

Great Britain.

Statistical records of the number of welded pressure-vessels made in Great Britain are not available, but as most of the important work, whether for land or marine purposes, has been done under the survey of Lloyd's Register, the Society's records at least give an indication of the extent to which welding has been applied in the pressure-vessel industry.

Since July, 1934, when the Society's "Tentative Requirements for Fusion Welded Pressure Vessels intended for Land Purposes" was first published, between 400 and 500 vessels, not including boiler-drums, have been constructed in accordance therewith. Many of these were intended for installation on board ships classed with the Society, others were intended for non-marine plant both at home and abroad. These vessels include heater-shells, air-receivers, caustic-liquor containers, digesters, high-pressure autoclaves, chemical-reaction chambers, condensers, evaporators, and other pressure-vessels forming part of oil-refinery plant, refrigeration-plant, etc. In addition, no less than 75 fusion-welded boiler-drums have been made under survey. These include drums for boilers intended for service in Great Britain as well as abroad. A number of these drums are for service in Australia.

As an indication of the present activity in the welding of boiler-drums in Great Britain, Lloyd's Register have, at the present moment, about 70 welded drums under survey during construction and intended for land purposes. These drums range in size from 10 feet 6 inches long by 3 feet internal diameter by $\frac{7}{8}$ inch thick, to 36 feet long by 4 feet 6 inches internal diameter by 2 inches thick.

It is of interest to note that eight oil-tankers are at present being built in America to the classification of Lloyd's Register of Shipping. Each of these ships is being equipped with water-tube boilers working at a pressure of 450 lbs. per square inch and having fusion-welded steam- and water-drums.

The Society's surveyors have also had to deal with certain special types of boilers and pressure-vessels which embody a considerable amount of welding in their construction. These include the Loeffler boiler intended for a working pressure of 2,130 lbs. per square inch at present being installed at the Brimsdown power-station. A similar boiler with a working pressure of 130 kilograms per square centimetre was recently fitted in the S.S. *Conte Rosso*, an Italian-owned passenger ship classed with Lloyd's Register. Other welded pressure-vessels surveyed during construction are the Velox steam-generator, Sulzer monotube boiler, special whale-oil boilers, and calandrias intended for sugar-refineries.

These concluding remarks will suffice to illustrate the development which has taken place in the application of welding in the pressure-vessel industry. In many of these cases welding is not merely an efficient substitute for riveting, and it is safe to say that without welding some of the latest advances in steam-generating plant could never have been made. *Fig. 11* (facing p. 661) shows a typical pressure-vessel having fusion-welded longitudinal and circumferential joints.

ACKNOWLEDGEMENTS.

The Author wishes to thank the Committee of Lloyd's Register of Shipping for their permission to publish certain information, and also Mr. H. N. Pemberton, a member of his Staff at Lloyd's Register of Shipping, for his assistance in the preparation of the Paper. Acknowledgements are also due to Messrs. Babcock & Wilcox, Ltd., of Renfrew, Messrs. John Thompson (Wolverhampton), Ltd., and Messrs. Sulzer Bros., of Winterthur, for the use of certain photographs.

The Paper is accompanied by three sheets of diagrams and by twenty-two photographs, from some of which the Figures in the text and the two half-tone page-plates have been prepared.

Discussion.

The AUTHOR showed a number of lantern-slides illustrating his aper, one of which was reproduced as Tables IX and X. Those The Author.

TABLE IX.—EFFICIENCIES FOR FUSION-WELDED BUTT-JOINTS FOR PRESSURE-VESSELS.

Conditions.	Allowable joint-efficiency: per cent.		
	Class 1.	Class 2.	Class 3.
pproved manufacturers' routine tests (a), heat-treatment, X-rays	90 (unfired vessels) 85 (fired vessels)	—	—
outine tests (b), heat-treatment . . .	—	80	—
outine tests (b), no heat-treatment . .	—	75	—
outine tests (c), heat-treatment . . .	—	—	70
outine tests (c), no heat-treatment . .	—	—	65
ydraulic test only	—	—	60

TABLE X.—EFFICIENCIES FOR FORGE-WELDED JOINTS FOR PRESSURE-VESSELS.

Class 1.	Fired vessels: 80 per cent.; unfired vessels: 85 per cent.
her vessels	Shell-thickness $\frac{5}{8}$ inch and above; 75 per cent. Shell-thickness less than $\frac{5}{8}$ inch; 69·5 per cent.

ables were reproduced from "Requirements for Fusion Welded ressure Vessels."[1] The formula governing working pressure in hich those figures for joint-efficiency were intended to be used as the same for all classes of vessels. The omission of heat-eatment in the case of Class 2 vessels was strictly limited to rtain vessels, depending upon the ratio of the internal diameter the cube of the shell-plate thickness. The reason for the figure

[1] *Loc. cit.*

The Author. of 69·5 per cent. for the efficiency of a forge-welded joint fo shell-thickness of less than $\frac{5}{8}$ inch was that for some time Lloy Register had included in the Rules for Air Receivers a formula calculating the thickness of forge-welded shell-plates, and, in or that there should not be any appreciable difference between results obtained by that formula and by the new general formu the figure of 69·5 per cent. was assigned.

It was necessary to point out that routine tests (*a*) included all-weld-metal tensile test, a joint tensile test, impact-tests, a bend-tests. Routine tests (*b*) included a joint tensile test, a be test, and a nicked-bend test. Routine tests (*c*) were similar routine tests (*b*), but the test-requirements were slightly eas For full details of those tests reference should be made to published Requirements.

Sir William Larke. Sir WILLIAM LARKE remarked that the Author stated that we ability of the plate was an important factor, which might rece more attention. Sir William could assure him that it was receiv a great deal of attention in several laboratories belonging to th who were interested in producing plates. In the absence from meeting of the President of The Institution, who was also Presid of the Institute of Welding, Sir William would give some informat as to the progress made by that Institute in discharging the manc which had been given to it by The Institution and fourt other bodies who were responsible for the Welding Symposium l in 1935. At the end of June, 1936, the Institute of Welding been asked to undertake the co-ordination of existing research the initiation of new research, and the Council of that Instit under the guidance of Sir Alexander Gibb, President Inst. C had accepted that mandate. To discharge its responsibilitie that connexion had involved, as might be imagined, the securin certain financial support, but developments had reached a s where additional financial support had been secured and the Insti had, he hoped, been placed in a position to commence to disch the mandate entrusted to it in the course of the next few mon To carry out the full task allotted to it would naturally, howe require many years of work.

He thought that all who were interested in the welding w agree that the Author had done much to encourage and develop application of welding to pressure-vessels; had it not been for Author, the application of welding to pressure-vessels in G Britain would have been in a very much more backward state. Author had had the courage to apply his knowledge to prac conditions where he had had to accept personal responsibility the results, and it was heartening to learn that there had l

o failures in service among the very large number of pressure-essels which had been welded under the survey of Lloyd's Register. Sir William Larke.

Engineer Rear-Admiral A. G. Crousaz said that with a little skill nd ordinary materials a joint of some sort between certain metals ould readily be made with consistent regularity. Its general utility nd economy in use had caused welding to be widely adopted, and as a esult welding plant had soon been in use in the majority of metal-orking shops; it might have been realized that, as normally ractised, electric and oxy-acetylene welding did not produce a joint hich was as strong as the parent-metal, but at any rate it was uite effective for a large amount of work, and it was cheap. Joints ad, however, failed, with disastrous results, and efforts had then een made by those who had need of really good welds to improve ne quality of the resulting joint. Research had been commenced ith that object in view, and it was not long before the technique ad advanced sufficiently to justify the expectation that arc-welding ould be made sufficiently reliable to be used for pressure-purposes, rovided that certain procedure was followed. The Author had tated what the result had been in America, and he was now intro-ucing the subject to those interested in Great Britain. It had to e admitted that the new craft had been received with ultra-onservatism. Such feelings could be allayed only by study and nderstanding, and it was a matter of congratulation that an ifluential body had taken the trouble to probe the subject as deeply s the present state of knowledge allowed, and had then had the ourage to take the first important step in the official recognition of ne value of welding in the fabrication of large pressure-vessels. Engineer Rear-Admiral Crousaz.

What had hitherto been the objections to the use of welding for nportant purposes? In the first place, no doubt, the knowledge f the failures which had occurred; but, as was recorded in the aper, those failures were in connexion with welds made before the ays of research and the issue of suitable codes. In the second lace, it had frequently been pointed out that any electrode or ller-rod used would appear in the welded joint as cast metal, which *so facto* could not be as good as the rolled or wrought metal to hich it was attached, and it was therefore a weak and potentially angerous spot. Steel castings had always been viewed with a rtain amount of suspicion, and even to-day there was plenty of idence in the modern steel casting to justify a certain lack of nfidence. The defects which arose, however, were largely due to ze and complication of shape, and a small casting of suitable com-osition could possess very good properties, little distinguishable om a corresponding forging, provided that the metal was reason-ly clean and was suitably heat-treated. There were many motor-

Engineer Rear-Admiral Crousaz.

cars with highly-rated engines of which the crank-shafts were ste castings. To be successful, such castings were bound to be ma under rigid rules and inspection, and the same thing was found be true of steel welds; research indicated that, provided th welds were made under carefully-controlled conditions and we subjected to rigid inspection, they could be relied upon to give go service. The filler-metal, although cast, would be found to ha properties generally comparable with those of the parent-metal the latter were suitably chosen; only in the resistance to fatigue w there sometimes any striking deviation in that respect.

The Tables in the Paper gave fatigue-limits for the weld-met which were generally distinctly good and not a great deal bel what the ideal weld-metal for boiler-plate should possess, b occasionally samples were tested which were not so good in th respect, and so far the deficiency had not been adequately explain Further research as to the fundamental cause of such low values fatigue resistance was considered to be necessary, since, although t resistance was probably quite adequate for such structures as boil drums, if welding were required to be subjected to repeated alt nating stresses it would naturally be desired that the fatigue-resi ance of the weld-metal should be reliably and consistently high.

In the Introduction to the Paper riveted joints in pressure-vess seemed to be treated very leniently; the Author omitted to po out the troublesome leaks that sometimes arose at the seams, caustic embrittlement in boiler-drums and the fact that in any c riveting became impracticable in certain circumstances, solid forgi or welded seams being essential. Whether solid-forged pressu vessels were better than welded vessels was, perhaps, a matter opinion, but it was certain that the former were more expensi Leaks in longitudinally-riveted seams were no doubt due to racking strains produced when a drum was placed under pressu and to the same cause might be ascribed the failure of the wrapp plate in a position adjacent to the butt-strap, the latter causin serious increase in local stress.

There appeared to be little fault to find in general with the te of Lloyd's requirements for the fusion-welding of pressure-vess which gave evidence of study and thought having been expendec the subject. Doubtless opinions would differ on certain aspect the code, but only experience would decide which was right. I dentally, it might interest the Author to know that, although l used, a specification for the arc-welding of pressure-vessels formulated by the Admiralty as early as September, 1932, but conformity with the usual practice, it had not been published.

In the section headed "Welded Joints," the Author stated

t had to be recognized that the welded joint created a hard spot in the structure. That was no doubt undesirable, but the question arose as to whether that statement was necessarily true. The answer seemed to be supplied by Table IV (facing p. 638), wherein were recorded figures for all-weld-metal which was certainly softer than the hardest grade of parent-metal allowed. It was certainly undesirable that the weld-metal should be unnecessarily hard, and for that reason the claim so often made that the tensile test-piece made with a certain electrode invariably broke through the plate and not through the weld, could not be admitted *per se* as being a good advertisement for the electrode. It was quite conceivable that it might be a better joint for the intended purpose if the test-piece failed through the weld-metal. It might be pointed out that hard weld-metal ought not to be detrimental in itself from the point of view of fatigue, since it should normally have a higher fatigue-limit than if it were softer, but it was undesirable on account of the concentration of stress that it would produce on the adjacent parent-metal if the latter were softer than the weld-metal. Engineer Rear-Admiral Crousaz.

The Author had said that the fact that the Izod test could not be directly correlated with other types of test should not necessarily be considered as detracting from its value. Admiral Crousaz was of the opinion that that property of the notched-bar test was the very reason for its existence and usefulness. It indicated in a relatively simple manner some property of the material which was known by experience to be important, but which in certain cases could not be made evident by any other of the well-known tests; hence the necessity for the retention of the Izod or some other equivalent test.

It was of interest to learn that the Author had doubts as to the value of the specific-gravity test, for considerable use of it had been made in the past. Before the application of X-rays to welding, it could be said with confidence that the specific-gravity test was of use in giving a rough idea of the freedom of the weld-metal from gas-pockets and slag-inclusions, but in recent years that function had been far better exercised by radiography, and he fully concurred in the Author's views in that respect. It might be observed that weld-metals with a specific gravity fully up to the normal requirement of 7·8 had given radiographs which, on account of gas-pockets and slag-inclusions alone, would be accepted by no authority responsible for the safety of pressure-vessels.

Dr. W. H. Hatfield remarked that the classification "A.1 at Lloyd's" carried great weight, and he considered that Tables IX and X (p. 667) showed the real facts of the situation. In the absence of weld the full characteristics of the plate-material were admittedly Dr. Hatfield.

Dr. Hatfield. coming into play, but when a weld was employed the efficienc ranged from 90 per cent. to 69·5 per cent., according to the classi cation as regards the care with which the operation was done ar examined. In Table IX it would be seen that the absence of hea treatment had depreciated the efficiency of a joint to 75 per cent and no doubt the Author would admit that that was a pure arbitrary decision, but the question of residual stress was ve important. Dr. Hatfield had seen thicknesses of 4 inches of sol mild steel broken through from residual stress arising from t faulty application of the welding procedure. From the scienti point of view, that was extremely interesting. It was possible have residual stress up to the breaking-down point of the materi as a result of wrong procedure in welding. He would like emphasize that, and he would have felt more content with rega to the 75-per-cent. efficiency (where heat-treatment was not applie if the Author had said at the same time that such a procedure w permissible only where it was merely desired that the vessel shou hold together, as for example in the fabrication of parts which to the place of castings. It was only for such purposes that untreat welded structures could be permitted to go into service.

At the Welding Symposium held in 1935 he had defined a weld the art of joining together two pieces of metal so that the compositi and properties should become continuous and uniform. From t information given in the Paper, it was possible to judge how welding could approximate to that ideal, and it was quite clear th useful though the procedure was in many types of work, it was n possible to attain in a weld either uniformity of composition reliability approaching that which would be obtained in a rolled forged piece of metal. When an inclusion or two was seen in forging an investigation was demanded, but much greater imp fections were allowed, even under an X-ray examination, where weld was concerned. That should be borne in mind.

The cheapness of welding had been pointed out as being advantage, but to advocate a procedure because it was cheap w not a satisfactory argument. Further, he rather questioned whet Admiral Crousaz was correct in what he had had to say if he w referring to highly-stressed parts. If the relative values given the Paper were compared, it would be found that the difference price was more than counterbalanced by the lack of reliabili which the Author had so ably emphasized.

He was glad that Admiral Crousaz had pointed out the value the impact-test. Dr. Hatfield held no brief for the impact-test, it was clear that very low values from impact-tests were of necess obtained from welds, particularly in some circumstances ; he wo

also point out that, if a notched-bar test showed that the specimen behaved as a tough material, that material evidently had properties which a material which broke in a brittle manner had not. He doubted very much whether the Author was right in stating that it was possible to rely upon a weld with mild steel to give a value of ±11 tons per square inch in fatigue, and he thought that, from the standpoint of reliability, that value was placed too high. Dr. Hatfield.

Professor B. P. HAIGH drew attention to a statement on p. 622 that a riveted joint could fail only in certain well-known ways. He thought that the four ways which the Author tabulated were fortunately almost unknown, and that a fifth way, namely, failure by leakage, with all its consequences, was more familiar in practice. Over 15 years ago he had carried out tests in connexion with a riveted boiler-seam that had caused continued trouble from leakage. After the defective seam had been electrically welded, it was found to stand, without permanent strain or other sign of injury, tensions up to four times those which had previously caused bending and consequent leakage. The electrically-welded seam had remained in service since then. It was possible to learn a very great deal from experience of repairs, and he hoped that that possibility would not be neglected. Professor Haigh.

The Author referred to water-gas welding as well as to electric welding, and the figures quoted in the Paper seemed to draw a comparison. Professor Haigh thought that a sharp contrast ought rather to be made. The two processes could be compared, perhaps, from the point of view of the user, because they served similar functions, but from the point of view of the manufacturer and the metallurgist there could be no sharper contrast than that between the two processes. Whereas water-gas welding commonly suffered from the excessively long-continued heating which it received, electric-arc welding, on the other hand, was liable to suffer from the excessively short period of time which was given for the heat to dissipate away from the bead of metal. The figures given in the Paper for water-gas welds indicated very satisfactory yield-points. He believed that they were about the highest which could be attained, and that as a rule the value would be a little lower, because a steel had to be chosen which was immune against the long-continued period of heating, especially in the case of thick plates; it was necessary to bear in mind that the period required to heat or cool a plate varied roughly with the square of the thickness. The water-gas weld therefore suffered on that account, especially in the case of thick plates. The severe danger that existed of deterioration from the long-continued heating required for welding thick plates would be appreciated by those who had tested such plates in a testing machine

Professor Haigh.

and had seen the ductility (as judged by elongation) decrease as th thickness increased.

In the case of electric welds, however, nearly all the difficulti arose from the rapid cooling, and if it were possible to control th rate of cooling and to delay it to suit the kind of steel being dea with, much less would be heard of cracks at the bottom of the V He would like to suggest that, when electric welds were being mad on new work, the method of heating the plates before they we welded should be adopted. In repair-work it was customary place a brazier very close to the job. In new constructio although it might not be necessary to go so far as that, he thoug that much might be done by combining water-gas or oxy-acetyle heating in order to heat the plate before welding, as in that case t dissipation of heat would be much less rapid.

There were in Great Britain a fair number of fatigue-testi machines large enough for testing welded plates up to 1 inch thickness, and those testing machines, although perhaps not qui large enough for the biggest jobs, did at least work many tim faster than hydraulic testing-machines. The testing-machines whi had been developed recently on the Continent, and which we operated hydraulically, seldom, he believed, worked at more th 200 strokes per minute, whereas electric fatigue-testing machin taking welds up to 1 inch in thickness could operate at 3,000 strok per minute.

Mr. Schuster.

Mr. L. W. Schuster deprecated the use of borrowed terms in t Paper. "Girth-seam," however concise it might be, was hardly recognized term in Great Britain, whilst "coupon" meant no mo than a test-sample such as was commonly incorporated with forging or a casting.

The Author was in charge of the engineering department of organization which had wide international interests, and in a stu of the rules which the Author gave it would seem that he w inclined to be governed by the necessity of adopting a unifo standard which could be applied in other countries in competit with rival organizations. Although Great Britain had undoubte learned much from other nations, Mr. Schuster thought that Brit engineers could claim to be in advance of all others in develop an electrode to give high-grade weld-metal.

Although it was true that the strength of a riveted boiler rated on the assumed tensile strength of the seams, perhaps insu cient stress was laid in the Paper on the fact that the relative ten strength of the weld- and parent-metals was not the true criter in rating the efficiency of a seam. A boiler did not explode ow to the attainment of the breaking strength of the material,

failed owing to the slow development of a crack. He had seen a very large number of cracks in the plates of boilers, but in no instance had the metal contracted as it did in a tensile test. The governing factor to be provided for, therefore, was failure due to some form of fatigue, even if it were corrosion-fatigue. As a welded seam of some length could hardly be expected to be without some small discontinuity, which was bound to weaken the weld-metal in fatigue, in any consideration of the resistance of a welded joint a substantial margin ought to be allowed between the fatigue-values of the two metals. Any reference to an efficiency of 100 per cent., although it might be well understood by boiler-engineers, was apt to be misleading. As a stress range of ± 15 tons per square inch was obtainable in boiler-plate, he would suggest, as a result of the study of test-figures, that a more likely figure for the ratio between the fatigue-strengths was 75 per cent., as compared with the 85 per cent. mentioned on p. 645 of the Paper. At the bottom of p. 627 the Author referred to "serious overheating . . . which cannot readily be rectified by subsequent heat-treatment . . ." As the Author did not recommend a heat-treatment at a higher temperature than about 600° C., that wording would seem to require modification. Mr. Schuster.

On p. 639 the Author referred to the possible use of the yield-point as a basis in the design of boiler-drums. Since the first essential in the design was to ensure an ample resistance to fatigue, and as there was no direct relationship between the yield-point and the fatigue-stress, and no evidence that an occasional rise of the stress beyond the yield-point lowered the fatigue-resistance, Mr. Schuster would like to point out that, for a boiler-drum working at the temperature of saturated steam, the use of the yield-point could have only a slender justification. The required test-figures and the designed factor of safety provided ample security against appreciable deformation, while, as long as the working temperature and stress were below those for appreciable creep, a comparatively low yield-point might even offer an advantage, in so far as it tended to relieve stress-concentrations. The Wöhler test was mentioned on p. 639, for testing welded joints. Although a Wöhler specimen might be suitable for testing the weld-metal, the junctions were not truly represented; for testing a welded plate the complete surfaces of the joint were best tested by the use of unmachined specimens, in a machine setting up alternating bending and arranged to give a constant bending moment. That had the advantage of reproducing the conditions of a boiler-plate in service.

The Author suggested on p. 649 a figure of 30 per cent. for the reduction in area. Mr. Schuster had for some years stressed the

Mr. Schuster. fact that, owing to porosity, the test-figure for the elongation weld-metal was not to be relied upon, and, for that reason, the figu for the reduction in area became of enhanced importance. Abo 2 years ago, a Committee of the British Standards Institution h decided upon a figure of 35 per cent. for the reduction in area Grade A metal; that metal was not intended to represent the be grade available, and for that reason pressure was at the mome being brought to bear on the British Standards Institution to pr vide for a metal of a higher quality. For such an important u as in a pressure-vessel it seemed unwarrantable to revert to t lower figure of 30 per cent.

The Author stated on p. 649 that either the angle of bend or t percentage-elongation demonstrated the capacity of the material withstand deformation without fracture. Mr. Schuster suggest that the word "material" ought to be replaced by the wor "welded joint"; the sentence appeared to refer to the weld-met whilst measuring the angle of bend certainly did not demonstra the capacity of the weld-metal to deform. Although doubtless t Author's intention had been to limit the application of the paragra to cases in which there were rigid test-requirements, in order ensure that the mechanical properties of the weld-metal and t plate were tolerably similar when the strain imposed might reasonably uniform along the joint, the statement was given as generality. In the practical range of welded joints, however, t properties of the two metals might be very different, and, with bend of 180 degrees, the strain in the weld-metal might in fact negligible, nearly the whole of the deformation being set up in t plate. In a recent test the specimen had been bent to very nea the same diameter as that referred to by the Author, but the stret had taken place largely in the plate and the elongation in the we metal was only 11 per cent., as compared with 30 per cent. stip lated on p. 651 of the Paper.

He did not consider that the two forms of bend-test mentioned the top of p. 649 were comparable, as was suggested. If the test w made merely to determine whether the joint as a whole would wi stand deformation, it was difficult to imagine anyone wishing measure the elongation. If, on the other hand, the Author intend "ductility" by the word "deformation," Mr. Schuster sugges that the Author would obtain extremely inconsistent results fr measuring the angle of bend. Although measuring the elongat was a somewhat cumbrous proceeding, which would preferably avoided, there was no doubt that, in the general testing of we metal, it served a very useful purpose. The measurement c stituted a much more ready and cheap method of determin

ıctility than the method of preparing a sample of all-weld-metal Mr. Schuster.
ıd making a tensile test on it; further, it was frequently
ıpossible to cut out a true all-weld-metal sample from a welded
int, and a specially-prepared all-weld-metal sample might contain
etal very different from that in the joint.

The Author had not mentioned one important use of the test. It
ıould always be kept in mind that the examination of the surface
' a bend-specimen provided an excellent means of judging the
ıality of the weld-metal in regard to porosity and small inclusions,
ıd as the outside surface of the weld could be put into tension that
eans of examination had certain advantages not even possessed by
ı X-ray examination. The British Standards Institution Specifica-
on, which was intended to cover welds for all purposes (including
ructures not subjected to pulsating stresses), was perforce lenient
its requirement, and the omission of the mention of that test in
ıe Paper might give the impression that the Author could afford
be more lenient still.

On the same subject, he was not at all happy about the statement
ade by the Author on p. 651, that 30 per cent. elongation repre-
nted high-class weld metal. He had on many occasions obtained
ı elongation of over 100 per cent. on a standard specimen of high-
ass weld-metal, and, if a metal would give an elongation of only
) per cent., it would probably be due to the opening-up of imper-
ctions. In view of the fact that there had been a great improve-
ent in the quality of electrodes since the figure had first been
ecified, it would seem unwise to adopt such a low figure. The
itish Standards Institution specified 40 per cent. elongation for
ass A electrodes, and the specifying of a lower figure gave the
ıpression that an inferior metal was good enough for a welded
iler. It ought to be recognized that, during the development of
sion-welding, there had been a natural tendency to deal sympa-
etically with manufacturers in overcoming their difficulties, and
r the time being to accept a type of fault which would not be
lerated in mild-steel boiler-plate. With regard to the last two
es on p. 650, he suggested that the Author did not intend that the
sts should be alternatives, so much as that the results should be
dged respectively by the angle of bend and the percentage-elonga-
n, and he would suggest that the words "angle of bend" ought to
replaced by "internal diameter," for the reason already given.

The statement on p. 652 that toughness or brittleness might in
rtain circumstances be revealed in a tensile or bend-test would
rdly seem to be correct, although a converse statement to the
ect that ductility was revealed in an Izod test would often, but
t always, be correct. Although it was true that, with a correctly-

Mr. Schuster. tempered alloy steel, there might appear to be some relation betw the respective test-results, the fact that when the steel was correctly heat-treated the relationship broke down, showed t toughness and ductility were independent properties. That further shown by the fact that a carbon steel giving excel ductility might be quite deficient in toughness, if that word w interpreted to mean the resistance of material to the spread c crack. Moreover, if by "toughness" the Author had intended imply that property of the metal which allowed it to accommod itself to concentration of stress at a notch, then Mr. Schuster wo be all the more disinclined to agree with him.

In a riveted boiler cracks were often detected before they spr sufficiently to become dangerous. On p. 653 the Author stated t ". . . crack-detection in welded joints is a matter of conside difficulty"; if that were to turn out to be common experienc would emphasize the importance of ensuring that the weld sho have a high resistance to the inception of a crack. In the paragraph the Author stated that ". . . it has yet to be pro that the full strength of a joint in a welded pressure-vessel is unc affected by very small areas of local metallurgical imperfectio Mr. Schuster would point out that it was just because of presence of those small imperfections, such as would open ou a bend-test, that weld-metal had a lower fatigue-range than tha a mild-steel plate.

It was stated (p. 654) that the impact-test might be regarded check on heat-treatment. That view had often been expressed, it hardly seemed to stand analysis. It was perfectly correct w other mechanical tests were not made, but, if all other tests satisfactory, there seemed no reason for ordinary purposes wishing to check the heat-treatment. The heat-treatment was r to secure satisfactory mechanical properties, and the test was r because it gave knowledge of the material that was not given by c tests. He agreed with the Author's statement that he did require a microscopic examination for the reasons just given. general, such an examination was of use only as an alternativ making mechanical tests, and not as an addition to them. It however, useful to reserve the right to make the examinatic assist in coming to a decision on some borderline test-result.

Mr. Davy. Mr. C. H. Davy said that the Author had dealt to some ex with the question of classification of pressure-vessels, stating type of weld to be used and the percentage-strength permitted different classes of vessel and methods of welding. The mere div of pressure-vessels into fired and unfired was scarcely sufficie determine the class of welding which was required. A boiler-

ntained steam and water, and as the water in the drum was at Mr. Davy.
turation-temperature, the energy stored in the vessel was con-
lerable. The same was the case with a vessel containing a liquefied
s. Some further subdivision appeared to be necessary to indicate
the designer exactly the class to which a welded vessel belonged.
raised that question as he continually received inquiries from
sign-departments asking direction as to the class to which a certain
pe of vessel had to be relegated.

The Paper seemed to point to fatigue-resistance as being the major
terion by which the value of a welded pressure-vessel was to be
lged, and he was in through agreement with the Author on that
int. There was, he thought, only one difficulty connected with
igue; namely, that at the present time it was not possible to
ntemplate fatigue-tests as a means for routine acceptance. What
uld be done was that actual full-scale vessels could be tested under
igue-conditions, so that the fatigue-value of the structure as such
uld be appreciated; it was also possible for a survey body or for
manufacturer to carry out a number of fatigue-tests on welds
de under standard conditions, which could be compared with the
pe of welds which the manufacturer normally produced. If the
igue-resistance of welded joints were compared with the fatigue-
istance of riveted joints—the comparison with an undrilled boiler-
te was scarcely fair—it would be found that on subjecting both
pes of joints to fatigue-tests of the constant-bend type, a figure
about ± 5 tons per square inch would be obtained for the riveted
nt and about ± 11 tons per square inch for a good weld. Further,
en welds which had been shown by X-ray examination to contain
ny mechanical faults, gave values the least of which was ± 5 tons
r square inch.

The important question was how to apply the knowledge obtained
fatigue-tests, and in that connexion he suggested that X-ray
amination of the welds was the important link. It had been shown
ite clearly in the Report of the Welding Research Committee of
Institution of Mechanical Engineers [1] that welds which were free
m mechanical defects gave the best results under the effect of
igue, and that mechanical defects had a great bearing on the
igue-resistance. It was undisputed that X-rays would show up
chanical defects such as lines of non-fusion, porosity, and slag-
lusions, and thus radiographic examination provided a means
which it was possible to appreciate with some degree of certainty
fatigue-value of welds, not only in test-plates but throughout the
ole of any welded pressure-vessel.

[1] Proc. Inst. Mech. E., vol. 133 (1936), p. 62, conclusion (3).

Dr. Reeve.

*** Dr. Lewis Reeve observed that, on p. 637, the Author gav
chemical specification for steels which, he stated, "... will be fou
entirely suitable" for fusion-welding. Other factors, apart fr
chemical analysis, had a bearing on the welding quality of a st
in particular the method of de-oxidation employed in its ma
facture. In so far, however, as a purely chemical specification co
define weldability, there was nothing in the Author's analysis
which the steelmaker would take exception, with one import
qualification. He could not agree that there was any need to lo
the sulphur- and phosphorus-specification below the normal fig
of 0·05 per cent. maximum. Sulphur, and to a lesser degree ph
phorus, had sometimes been blamed for the production of weld
cracks, and there was no doubt that, with certain types of electro
abnormal sulphur in the weld-metal was a contributory factor in
production of such cracks. A far more important factor w
however, the type of electrode used, and if certain electrodes
present in favour were eliminated far less would be heard of s
welding troubles. In Dr. Reeve's experience too low a mangan
and too high a silicon-content in the weld-metal was by far the m
important factor leading to the development of welding cracks.
suitable electrodes were used he had found no difficulty in the weld
even of "tied" joints in steel containing sulphur and phospho
within normal limits. The introduction of a lower sulphur-
phosphorus-limit in the steel was unnecessary and would be
unjustified burden, not only upon the steelmaker but upon
consumer, who would have to pay for the production of such spec
quality steel.

In so far as sulphur had any significance it could be adequa
controlled by the electrode-maker in choosing wire for the ma
facture of electrodes. Such wire was produced in low-sulphur gra
by all wire-makers and would give satisfactory results if it w
correctly fluxed. It was unnecessary to expect the whole of a s
vessel to conform to a low-sulphur specification when the control
factor producing cracks was the quality of the weld-metal depos
by the electrode. If such weld-metal were abnormally sensitiv
sulphur, then either it ought not to be used, or the makers ough
have an extra low sulphur-content in the wire and fluxes. The c
for such material certainly ought not to be put upon the manufact
of the steel plate. It was not without significance that the Ameri
German, and Australian specifications quoted by the Author
corporated the normal figure of 0·05 per cent. (maximum) sulphu
shell-plates.

*** This contribution was submitted in writing.—Sec. Inst.

On p. 660 the Author referred to Dr. Reeve's Paper on "Internal Dr. Reeve.
resses in Welding and their Determination" and stated that he
as not aware that the method of mechanical stress-relief had ever
en applied in practice. Dr. Reeve would point out that one large
nerican manufacturer of arc-welded pipes normally incorporated
echanical overstressing of the pipe during the hydraulic testing
which each section was subjected. That was done deliberately
stress-relieve the butt-welded joint. Sufficient hydraulic pressure
as applied so that the resultant stress, when added to the large
ternal stress already present in and near the weld, produced slight
astic flow. It was claimed that the internal stresses were in that
anner largely eliminated. He would not, however, be prepared
suggest that such mechanical stress-relief could entirely replace
e present method of thermal stress-relief at temperatures of about
5° C., as described by the Author. Thermal stress-relief would not
ly give more uniform results but the temperature employed was
fficient to produce a tempering action at certain hardened portions
ich might occur in a butt-weld even in mild steel—for example,
ar the root as a result of the "sealing run" of weld-metal.
echanical stress-relief was of greater importance in connexion with
ge structural welding and in shipbuilding where thermal stress-
ief was virtually impossible.

The Author, in reply, pointed out that the Paper was in general a The Author.
atement of facts relating to the welding of joints in pressure-
ssels, and not just an expression of the Author's ideas on the subject.
ne of the main objects of the Paper was to show what had been
complished both in Great Britain and abroad, and to indicate the
portant factors on which practical experience had shown that
nt-efficiency depended. It was therefore somewhat surprising to
d Engineer Rear-Admiral Crousaz inferring that the Author was
w introducing the subject of welded pressure-vessels in Great
itain. Those who took pride in the industrial progress made in
eat Britain would hardly be prepared to acknowledge that the
bject was only now being introduced. In that connexion Admiral
ousaz himself acknowledged that the Admiralty had had their own
ecification for welded pressure-vessels as far back as 1932.

Sir William Larke's complimentary remarks were much appreciated,
t it was as well to emphasize that it had never been the Author's
sire to give special encouragement to any particular method of
anufacturing pressure-vessels. The fact was that welding had been
ore or less thrust upon him, and it was his object to ensure, so far
he could, that such development took place along sound and
ional lines. The regulations issued by Lloyd's Register of
ipping in 1934 had, he thought, given considerable assistance and

The Author. guidance to industry. Mr. Schuster had referred to the natu tendency on the part of inspecting authorities to deal sympathe ally with manufacturers in overcoming their difficulties during development of fusion-welding, and in case it might be inferred t the normal attitude of inspecting authorities was hostile rather t helpful to welding development, the Author was glad to say t the spirit of helpful co-operation was one of the underlying princi of the work of Lloyd's Register in all spheres of industry. It true, as Mr. Schuster had pointed out, that the work of Lloy Register was of an international character, but far from bein handicap to the formation of suitable standards—as Mr. Schu had suggested—the international aspect of Lloyd's Register, with enormous fund of information and its contact with developm throughout the world, was an asset which obviously could not denied. Mr. Schuster's claim that British engineers were in adva of all others in developing an electrode to give high-grade weld-m was, in the Author's opinion, nothing more than a commend patriotic sentiment. It was true that the covered electrode was introduced by British engineers, but he could hardly agree development in regard to the application of welding in Great Bri was ahead of that abroad. That was mainly due to the restricti reasonable and otherwise, which had been placed upon it.

With reference to the more detailed comments which had made by those taking part in the discussion, he had noted that Professor Haigh and Admiral Crousaz criticised his statements reg ing the types of failure of riveted joints, and that both referre the question of leakage. Whilst it was true that, as far as press vessels were concerned, a riveted joint which leaked badly migh deemed to have failed, nevertheless the four types of failure mentio on p. 622 were fundamental, and formed the basis of design of riv joints. Further, he would draw attention to the statement on p. to the effect that it was not the purpose of the Paper to conside the possible factors which might contribute to the failure of riv joints.

Several speakers had referred to the question of fatigue, an appeared necessary to emphasize that the figures for fatigue-endur quoted in the Paper were typical of many obtained from tests ca out for official purposes in connexion with vessels constructed u the survey of Lloyd's Register. In putting forward the figu ± 11 tons per square inch as a good average value for the W fatigue-endurance limit of high-quality welded joints, the Author simply drawing on the experience of Lloyd's Register in the testi welded pressure-vessels. Dr. Hatfield had expressed his disa ment with that figure, but he did not produce any authorita

idence in support of his contention that it was too high for the The Author. ality of work in question.

The Author was in agreement with Mr. Schuster, who pointed out at the Wöhler fatigue-test was not the best type of fatigue-test to ply to welded joints, but it was to be noted that, as shown in ble IV (facing p. 638), many tests had been carried out on machined specimens, both in direct tension and compression and so in alternating bending. The average figures for some of those sts were quoted in Table IV, and it would be noted that the figure 85 per cent. was a justifiable ratio between the fatigue-limit of a od-quality welded joint and that of the parent-plate. Some of ose tests had been made on the Continent using a machine capable testing specimens having a sectional area of $1\frac{1}{2}$ square inch in rect stress, the alternations having a frequency of 500 per minute. at would be of especial interest to Professor Haigh, who had ted that he understood that Continental machines did not work more than 200 fluctuations per minute. It was, however, interest- g to note that electric fatigue-testing machines, such as the Haigh achine, were capable of taking welded specimens up to 1 inch in ickness, and that they could operate at 3,000 fluctuations per nute. There could be no doubt that such machines were of the eatest value for the carrying out of fatigue-tests.

In the case of welded boiler-drums, he was inclined to think that due emphasis had been laid on the subject of fatigue-endurance. . Schuster had referred to boiler explosions which had been found be due to the slow development of fatigue-cracks, but such a tement was obviously unfair when used in the general way in ich Mr. Schuster had used it. No one would dispute that in tain types of boilers corrosion-fatigue conditions did occur, but was questionable whether such conditions ever occurred in the lded joints of water-tube boiler-drums, and in any case the uniform lindrical section of a welded drum, which incidentally could not be tained in a riveted drum, afforded the best possible resistance to rrosion-fatigue.

Fatigue-tests on small specimens and small sections of a welded nt were admittedly of some use, but such tests tended to magnify significance of small inclusions. Mr. Davy had pointed out that was possible to carry out fatigue-tests on full-size pressure-vessels ; h tests had been carried out on a number of occasions and the ults had proved that when ultimate failure did occur it was tiated at some point of stress-concentration, such as at a manhole in way of the holes, and not in the welded joint.

In regard to Tables IX and X (p. 667), it should be explained that the proposed Requirements of Lloyd's Register for fusion-welded

The Author. pressure-vessels, the omission of heat-treatment in the case of Cla vessels would be strictly limited, and he could assure Dr. Hat that it was not a case of simply omitting heat-treatment and qualifying for a joint-efficiency of 75 per cent. as compared with 80 cent. when heat-treatment was not omitted. As a matter of it was only proposed to allow the omission of heat-treatment in case of Class 2 pressure-vessels in which the ratio of inside diam of the cylindrical shell to the cube of the shell-thickness was in ex of 100.

The question of classification of pressure-vessels, mentioned Mr. Davy, was a difficult one. In the proposed revised Requirem of Lloyd's Register, whilst pressure-vessels might within ce limits be built to the requirements of any class, certain vessels w be obliged to fall within either the Class 1 or Class 2 category follows :—

Class 1 : Parts of boilers under pressure and fired pres vessels having a working pressure above 50 lbs. square inch.

Class 2 : (i) Fired pressure-vessels having a working pre less than 50 lbs. per square inch. (ii) Unfired pres vessels in which the working pressure × the shell-t ness was greater than 90. (iii) Unfired pressure-ve where the working pressure was greater than 250 lbs square inch. (iv) Unfired pressure-vessels in which working temperature was greater than 30 (v) Unfired pressure-vessels in which the shell-thic was greater than $\frac{5}{8}$ inch. (vi) All fusion-welded pres vessels for which the calculated working pressure o shell, according to the formula given in the Requirem required a longitudinal joint-efficiency above 70 per but not exceeding 80 per cent.

In regard to certain details of test-requirements, he agreed Mr. Schuster that 35 per cent. instead of 30 per cent. was a reasonable minimum figure for the reduction of area required all-weld tensile specimen; experience of high-class welding showed that figures much higher than 35 per cent. were gen obtained.

Mr. Schuster had misinterpreted the Author's reference a bottom of p. 627 in regard to "... serious overheating ... cannot readily be rectified by subsequent heat-treatment." heat-treatment referred to was not the stress-relieving tempe recommended as a normal routine procedure, but a full norma treatment which would bring about recrystallization.

In regard to the question of weldability of plate-material for The Author. essure-vessels, he was not prepared, in the light of knowledge at esent available, to advocate anything other than normal boiler- ality mild steel. The analysis given on p. 637 would be found tirely suitable for welding with most of the good-quality electrodes the market, and in that connexion it was necessary to say that w sulphur- and phosphorus-content in the plate were very desirable, ving regard to the somewhat high silicon-content often found in eld-metal.

In order to satisfy the special requirements for deposited weld- etal in the case of pressure-vessels, electrode-makers concentrated the problem of obtaining sound weld-metal free from porosity, d thoroughly shielded in a reducing media during deposition. hat necessitated the inclusion of de-oxidizing agents in the electrode, d he suggested that, where silicon was used as the principal -oxidizer, there was a danger of excessive silicon being contained the fusion zone where an intimate mixing of the parent-steel and eld-metal took place. One important effect of silicon appeared to a fairly high rate of contraction during solidification. That high te of contraction might cause such high stresses in the fusion zone at cracking would occur, and such an effect would be accentuated sulphur happened to be present in any other form than manganese lphide. Dr. Reeve had expressed the opinion that the normal lphur- and phosphorus-content for boiler-plate—namely, 0·05 per nt. maximum—was reasonable from the point of view of welding, ovided that suitable electrodes were used, and the Author would t disagree with him. Whilst it was unlikely that rolled-steel iler-quality plates would contain injurious quantities of phosphorus d sulphur, by virtue of the fact that they had been rolled success- lly, nevertheless it was well to realize that, from the point of view welding, the effect of phosphorus was to increase the sensitivity the steel to overheating and to render it " cold-short " and brittle. lphur, on the other hand, would render the steel " red-short "— at was to say, brittle when it was hot. So far as sulphur was ncerned, however, the Author would suggest that the important nsideration was not so much the quantity, but rather the condition which it was present in steel.

He could not allow Dr. Hatfield's criticism of welded pressure- ssels to pass without reply. It was true that, even to-day, there s much bad welding. It was, however, equally true that the lding of high-class pressure-vessels, under competent survey in cordance with a rigid code of testing and inspecting requirements, d reached a remarkably high standard, fully comparable from the int of view of reliability with the solid-forged drum. Dr. Hatfield

The Author. had stated that much greater imperfections were allowed in a welc vessel than would be tolerated in a forging, but in the case of hi class welding work that statement could not be support Dr. Hatfield was aware of the meticulous care and precision adop in the testing and inspection of welded joints in pressure-vessels, a the fact that such procedure revealed defects (which were eit rectified or condemned) was no argument in favour of solid-for drums which were not subject to the same testing and inspect procedure.

*** The Correspondence on the foregoing Paper will be publis in the Institution Journal for October, 1937.—Sec. Inst. C.E.

ORDINARY MEETING.

16 March, 1937.

Sir ALEXANDER GIBB, G.B.E., C.B., F.R.S., President, in the Chair.

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

Paper No. 5109.

"Kincardine-on-Forth Bridge."

By JOHN GUTHRIE BROWN, M. Inst. C.E.

TABLE OF CONTENTS.

HISTORY.

THE Firth of Forth thrusts itself westwards for some 50 miles from the general line of the east coast of Scotland, and since ancient times has acted as a barrier against the full development of the complementary interests uniting the now heavily-populated industrial areas in Fife and Clackmannan on the north shore with the agricultural districts of Stirling and the Lothians on the south. At the same time the cultural advantages of close association with Edinburgh have been made more difficult of access to the inhabitants of the Fife peninsula by reason of this estuary.

The coming of the railway era and the construction of the famous railway bridge over the Forth at Queensferry (1882–1889) brought about an immense impetus to the development of industry in east Scotland, and the importance of this bridge to Scotland generally has become more evident each year since its construction. Whilst the requirements of railway traffic across the Forth are adequately met by crossings at Queensferry, Alloa, and Stirling, the increasing need of road transport had until recent years not been given consideration.

Between Queensferry and Stirling, 25 miles to the west, the following crossings were available for road traffic:

(1) A ferry service at Queensferry immediately west of the railway bridge.
(2) A small passenger ferry at Kincardine, 14 miles west of Queensferry. No facilities are available for vehicles at this ferry.
(3) A somewhat primitive steam ferry-boat at Alloa, 18 miles west of Queensferry.
(4) The last and most important crossing is that of Stirling bridge. The amount of traffic crossing this bridge has increased almost threefold since 1922, with the result that considerable delay has, in recent years, been caused to traffic passing through the congested streets of Stirling.

With delay through this cause, added to a detour in the case of east-coast traffic of almost 50 miles, it is not to be wondered at that, with the increase in the number of motor vehicles after the war, agitation throughout central Scotland arose in acute form for the improvement of facilities for crossing the Forth.

In 1925 a report was prepared by the late Mr. Thomas Nisbet, and was submitted to the Ministry of Transport, examining the possibilities of bridging the Forth at Alloa, 5½ miles south-east of Stirling bridge, or at Kincardine, 13½ miles downstream from Stirling.

The river at Kincardine is 2,400 feet wide at high water, and this was the furthest downstream site where a bridge could be built at moderate cost—say under £400,000. Immediately beyond this the river widens out to an estuary about 3 miles wide, and from Kincardine down to Queensferry the bridging of the estuary would entail ten times that expenditure as a minimum.

Mr. Nisbet in his report, whilst admitting the greater traffic advantages of the Kincardine site due to its position further down-stream, was obliged to reject it in favour of the Alloa site mainly owing to the doubts about obtaining a foundation at a reasonable depth on the south side of the river. No immediate action followed the production of his report, but later the scheme for

a bridge at Alloa was again revived by the local authorities, as a result of which Messrs. Mott, Hay & Anderson were instructed to prepare plans and estimates for a crossing of the river at Alloa.

In 1930 the Convener of Fife County Council, The Rt. Hon. The Earl of Elgin and Kincardine, K.T., C.M.G., instructed Sir Alexander Gibb & Partners to report on the feasibility of a crossing at Kincardine. A scheme for a railway crossing under the river near Kincardine had been projected in 1890 and bores had been put down by the engineers, Messrs. Formans & McCall, of Glasgow, on behalf of the promoters, the Caledonian Railway Company. Copies of the boring journals were in the possession of Sir Alexander Gibb & Partners. These showed rock quite close to the surface on the north side of the river, whilst on the south side, beyond where a fault occurred at midstream, although no rock was obtainable at a practicable depth for founding on, there existed a belt of gravel from 30 to 50 feet below the ground-surface which appeared to be suitable for carrying piles. The scheme for a bridge was obviously quite practicable, and preliminary designs and estimates were accordingly prepared and submitted to the Local Authority.

A meeting was held in Stirling in June, 1930, under the chairmanship of Lord Ponsonby, then Secretary to the Ministry of Transport. The two bridge schemes were considered at the meeting, which was attended by representatives of the Counties of Fife and Clackmannan and of the Burghs of Dunfermline, Falkirk, Alloa, and Stirling. It was unanimously agreed that a joint report by the two firms of consultants should be prepared on the alternative sites for a bridge across the Forth at Alloa and Kincardine.

This report was submitted to the Ministry of Transport in August, 1930. Each scheme was based on a roadway 30 feet wide with two footpaths 6 feet wide, and having a swing-span providing an opening 100 feet wide for shipping. The report dealt in detail with each site from the points of traffic advantages, engineering and costs, both for 20-foot and 30-foot roadways. Based upon a 30-foot roadway with two footpaths each 6 feet wide, and including all necessary approach roads to link up with the existing roads on either side of the Forth, the estimates were :—

Alloa bridge	£418,000,
Kincardine bridge	£385,000.

The balance of the financial and traffic considerations, the report concluded, would clearly seem to lie in favour of the Kincardine bridge.

Although the erection of the bridge at Kincardine was of less direct local benefit to Alloa, the County Council of Clackmannan agreed to support a bridge at Kincardine, and thus was formed,

with the instigation of the County of Fife and the co-operation of the Counties of Stirling and Clackmannan, the Joint Authority responsible for the promotion of the scheme. Financial assistance was also contributed by the Burghs of Dunfermline and Falkirk.

A Provisional Order was promoted in 1930 for the bridge, the hearing before a Parliamentary Joint Committee taking place in Edinburgh during March, 1931. The scheme, as submitted to the Joint Committee, provided for a bridge 2,425 feet long, with a swing-span giving two openings each 100 feet clear for navigation, 25 feet headroom being provided elsewhere. In addition, the approaches linking up with the existing roads on either side of the Forth were as follows :—

> A south approach road 2,200 yards long from the Grangemouth—Stirling road to the south end of the bridge. This approach provided a 30-foot roadway and a 6-foot footpath.
>
> A north approach road 500 yards long from the Dunfermline—Alloa road to the north end of the bridge, also giving a 30-foot roadway and two 8-foot footpaths.
>
> A by-pass road of similar width and 700 yards long to avoid the narrow and tortuous roads through the town of Kincardine.

The only opposition offered to the scheme was by the Chamber of Shipping on behalf of the navigation interests. The river, it should be noted, is navigable to Stirling and is mainly used above Kincardine by vessels trading to Alloa dock for coal and to South Alloa with oil and timber pit-props. The maximum size of vessel passing the bridge-site is about 1,500—2,000 tons. The number of times that the bridge will require to be opened, according to present traffic, is from 30 to 40 per month, with a possibility of future increase. Agreement was finally reached on the basis of giving two openings for shipping each 150 feet clear, with a clear headroom above high water of 30 feet over this 150 feet width. The Provisional Order received the Royal Assent on the 8th July, 1931. The grant promised by the Ministry of Transport towards the cost of the Scheme was 85 per cent.

Contract drawings were put in hand immediately thereafter, and tenders were called for in December, 1931. Contractors were asked to quote separately for the foundations and superstructure of the bridge. Twelve tenders were received in all, of which the lowest was the combined tender of the Cleveland Bridge & Engineering Company, Ltd., of Darlington, as follows :—

Foundations	£125,149
Superstructure	£113,460
	£238,609

The next tender amounted to £254,905.

The economy requirements of the Government instituted at this period in 1931 prevented approval being given by the Ministry of Transport to the carrying out of this scheme. The project was accordingly suspended until 1933, when the improvement in the country's finances permitted the scheme to be revived. The amount of grant to be given by the Ministry was, however, reduced from 85 per cent. to 75 per cent. The raising of an additional sum of about £30,000 which this necessitated caused considerable difficulty and anxiety amongst the local authorities. Ultimately, however, in the autumn of 1933 the finances were satisfactorily arranged, and the scheme was finally approved by the Ministry in September, 1933. The Cleveland Bridge & Engineering Company were still prepared to adhere to their tender of £238,609, and were accordingly accepted for the whole of the bridge-work.

The ceremony of cutting the first sod was carried out in December, 1933, by Lord Bruce, and the bridge was formally opened to the public on the 29th October, 1936, by turning the swing-span into the road position. The opening ceremony was carried out by the Rt. Hon. The Earl of Elgin and Kincardine, K.T., C.M.G., Convener of Fife, C. E. Horsburgh, Esq., Convener of Stirlingshire, and the Rt. Hon. The Earl of Mar and Kellie, K.T., Convener of Clackmannanshire.

The duration of the contract was 2 years 10 months and the number of men employed at the site on the bridge averaged about 150, with a peak total of 210 towards the end of the contract. The whole of the work was carried out without a single serious accident to any of the men engaged.

The result of the construction of this bridge has been to provide all traffic through central Scotland with an alternative crossing over the Forth, which reduces the detour by Stirling bridge to a pronounced degree; savings of from 5 to almost 50 per cent. are made on routes covering traffic originating from east, west and south. The bridge can, in fact, be claimed to be a national asset by reason of the actual mileage savings which it provides. It benefits not only east-coast traffic, but gives also a straight-line route for west-coast traffic from Glasgow to Fife. Further, it opens up an attractive alternative route to Perth as well as a direct route for the south-to-north traffic *via* Lanark. A census taken at the bridge on the 6th November, 1936, showed that 1,246 vehicles had crossed between 6.0 a.m. and 10.0 p.m.

Site of Bridge.

The bridge is situated about 150 yards downstream of the passenger ferry already referred to. It was located to clear the ferry-pier, the shipping pier at Kincardine belonging to the L.N.E. Railway, and

Kincardine railway-station on the Alloa—Dunfermline branch of that company's railway. It is interesting to note that the 132,000-volt line of the Central Electricity Board passes diagonally over the bridge, the cables being carried 150 feet above high water by two large steel towers.

The width of the river at low water at this site is 1,500 feet whilst at high water it is 2,400 feet. The north shore of the river is defined by the railway company's pitched wall, whilst the south shore consists of an extensive area of flat and muddy foreshore or "saltings." It is believed that the river-channel at one time in its geological history was located over the site of these saltings on the south side, and the gravel beds 50 feet below the overlying mud give corroborative evidence of this having been the case. The river at the bridge site is tidal, with a range of 19 feet at springs. The tidal currents are fast, and as a rule the river is heavily charged with silt.

Bores at Site.

For the purpose of confirming the practicability of the design of structure which had been prepared on the results of the 1890 bores, a series of bores was put down on the line of the bridge. The record of these bores is given in Fig. 1, Plate 1, and they confirmed to a very close degree the results of the bores taken a short distance upstream in 1890. Generally it will be seen that on the northern half of the river the bed consists of from 2 to 10 feet of sand, gravel and broken rock, overlying grey sandstone of the carboniferous period. The planes of the rock-beds have a pronounced dip to the north, with the result that the successive beds of outcropping grey sandstone on which the piers were founded became tougher and harder as the work advanced into midstream. At the centre pier the outcrop of rock founded on was possibly 80 feet below ground-surface at the north shore, and as a result the rock at this place was of particular hardness. Bore No. 7 was on the site of the swing-span centre pier.

Although not shown in Fig. 1, Plate 1, two further bores were put down 30 feet and 50 feet south of the centre-line of the main swing-span pier in order to establish definitely that this important foundation was not on the edge of the fault which was known to exist near this place. This fault, which generally follows the centre-line of the river, has resulted in a throw-down of the whole of the rock from a point about 100 feet south of the centre pier of the swing-span. The bore at pier No. 12 (bore No. 6) clearly shows the broken and shattered nature of the rock due to this fault, and this, in addition to the greater depths to solid rock, necessitated the adoption of piled foundations for the whole of the southern section of the bridge.

Fig. 3.

KINCARDINE-ON-FORTH BRIDGE.

5109.

Fig. 18.

SWING-SPAN NEARING COMPLETION.

Fig. 23.

CENTRE PIVOT AND ROLLERS ASSEMBLED.

Particular reliance was placed on the bed of gravel which exists about 50 feet below ground-level on the south shore, and it was assumed in designing the structure that the piles would obtain a satisfactory set in this gravel, as in fact actually occurred.

It is gratifying to state that the bores provided an excellent epitome of the character and extent of the various materials comprising the bed and substrata. None of the materials exposed or passed through in the foundations during construction disclosed any necessity to revise the particulars given in the journals.

General Description of Bridge.

As will be seen from Fig. 2, Plate 1, and *Fig. 3* (facing p. 692), the bridge is a multiple-span structure and introduces various types of design and materials of construction.

Commencing at the north or Kincardine end of the structure, the following spans are provided :—

(1) Three continuous spans of 62 feet 6 inches over the L.N.E. Railway property, these being built on a curve of 520 feet radius to connect with the alignment adopted for the north approach road. This road had to be located within narrow limits to avoid excessive demolition of property.

(2) Seven steel spans of 100 feet constructed as a system of cantilevers with 50-foot suspended girders in alternate spans. This type of construction, besides being economical and convenient for erection, lent itself admirably to the adoption of the curved or arched shape of girder which is a characteristic feature of the design.

(3) A steel swing-span, 364 feet overall, of the through type symmetrically balanced on a centre pier at midstream. This structure is protected from shipping when in the open position by a timber jetty 470 feet long and 50 feet wide.

(4) Seven steel spans of 100 feet, as in (2).

(5) Nine reinforced-concrete spans of 50 feet with an arched underside to the beams similar to that adopted for the steel spans.

(6) A piled reinforced-concrete viaduct 265 feet long.

The actual total length of the bridge as constructed is 2,696 feet, with a clear headroom under the swing-span of 30 feet at high water. The roadway-level at the centre of the swing-span is 46·5 O.D.

Particular care was taken to provide a well-considered and harmonious vertical sweep of roadway over the length of the bridge

in view of the considerable rise in midstream and the greatly foreshortened effect which could be obtained by viewing this ½-mile-long structure from either shore. Due to the existence of the L.N.E. Railway, which required the road to be about 22 feet above rail-level at this place, it was not possible to provide an entirely symmetrical curvature in both directions. Actually, the south side roadway is considerably lower, and a reverse curve is introduced at the viaduct to sweep gradually on to the south approach-road gradient. The maximum gradients are 1 in 40 on the south side and 1 in 38 on the north.

The appearance of the vertical curvature given by the completed structure is satisfactory and justifies the care which was devoted to this matter during the design and fabrication of the superstructure. The foreshortened view observed by the public when walking or driving over a bridge is not easily visualized from the drawing of a bridge-elevation and the unfortunate results seen in some structures indicate that this aspect has occasionally been overlooked.

Setting-Out.

One of the first matters to be dealt with was the setting-out of the bridge structure to permit of construction being proceeded with simultaneously on both sides of the river. Two permanent stations were established about 2,000 feet apart, one on either shore between piers 2 and 3 and 21 and 22, and on the exact centre-line of the bridge. The distance between these stations was determined by triangulation from a base-line on the north side. The L.N.E. Railway embankment was selected as a suitable site for this base.

Whilst reasonable accuracy was desired, no exceptional efforts to obtain absolute accuracy were considered necessary, as in the event of a few inches error arising in the closing length at midstream this could easily be rectified in the 19-foot-wide piers on either side of the swing span. The base line 2,137 feet long was measured with a 500-foot steel band under a constant pull of 10 lbs., adjustment being made for temperature. Angles were read by means of a 3½-inch diameter internal-optical-micrometer theodolite. The distance thus determined was 1,993·74 feet, and from this the setting out of the bridge-piers on either side was undertaken. A year later, after the staging had been completed to mid-river leaving a 150-foot gap only, the closing distance was checked by direct measurement and was found to be about 1 inch too wide. The measurement across the gap was obtained during calm-weather conditions, experiments having been made on the shore to determine the correction for sag. This slight discrepancy of 1 inch was adjusted without

difficulty in piers Nos. 10 and 12. For the levels required, readings were transferred across the river by reciprocal methods using a precise level. Here also a check was possible across the navigation-opening at a later date.

TEMPORARY BRIDGE.

To provide and maintain a structure with an effective life of 3 years, built ¼ mile from either shore into the turbulent and swift-flowing waters of the Forth, it was evident that a fairly substantial design would be the most economical in the long run, in view of the duration of service required and the heavy loads to be carried over it. The contractors, acting on this policy, produced a substantially-designed structure (*Figs. 4*, p. 696) consisting of 12-inch by 12-inch piles and 12-inch by 6-inch head-timbers, with 18-inch by 7-inch steel joists to carry the standard-gauge crane and wagon track. Ample steel angle-bracing was provided to stiffen the structure.

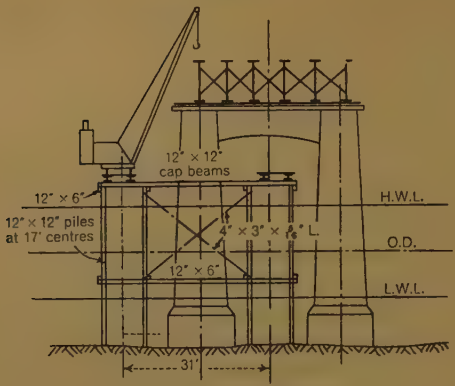
The north shore staging consisted of two standard-gauge tracks at 31-foot centres. The upstream track was built between the twin-cylinder piers exactly on the centre-line of the bridge, the downstream track being clear of the downstream cylinder. During construction four piles forming a bent were temporarily tied together in proper transverse position with walings and lowered into the river bed. Thereafter the piles were driven to a suitable set by means of a McKiernan-Terry hammer seated on top of the pile.

This method of construction was rapid and good progress was made, the 960 feet involved in the section from the north shore to the centre permanent jetty being completed in 6 months by August, 1934. The piles in every case were driven to refusal on to rock and the points of these when plotted gave a good indication of the rock-level to check against the borings. Near pier No. 10 the cover of soft material over the rock thinned down to 1½ foot and difficulty was experienced in obtaining a grip for the temporary piles in the river-bed.

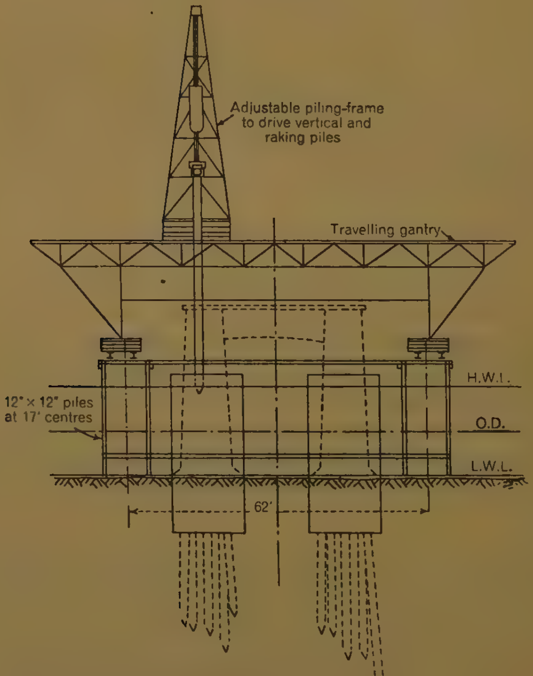
On the south side a different spacing and arrangement of staging was adopted to suit the site-conditions, and the necessity of driving permanent piles for all the foundations on this side of the river. A pile-casting yard was established clear of the saltings on more solid ground about 150 yards west, actually on the side of the access road to the passenger ferry. A curved approach track led from this pile-casting area to the line of the bridge.

Owing to the nature of the saltings at this place with a 45-foot depth of soft muddy clay, it was found that temporary wooden piles 25 feet long, when driven for the crane track, gave practically no

Figs. 4.



SECTION AT PIER No. 5.



SECTION AT PIER No. 18.

Scale: 1 inch = 40 feet

Feet 10 5 0 10 20 30 40 feet

TEMPORARY STAGING.

support at all to the crane load. To avoid the use of piles about 65 feet long, which would require to be driven to the gravel bed, the contractors adopted the use of a close layer of timber sleepers laid on the rank grass which covers the saltings. This cover, if unbroken, offers considerable resistance to settlement, and by means of a crib-work of timber laid on top of the sleepers a crane track was provided which remained practically unaltered throughout the duration of the contract, and carried safely the whole of the materials for the south-side construction.

The staging along the line of the bridge was located on this side of the river on the outside of the piers, the tracks being at 62-foot spacing to carry the overhead travelling gantry (*Figs. 4*). On top of this gantry a piling-frame was erected which could be moved transversely, rotated in any direction, and tilted to drive batter piles.

This timber structure was commenced in July, 1934, and good progress was made with its erection. As the staging advanced into the river the normal design with vertical timber piles was adopted. The ends of the piles were driven with blunt square ends and in certain cases projecting timber blocks were also fastened to the piles to increase their resistance to settlement in the soft material.

During February, 1935, when the temporary staging had reached the site of pier No. 13 and a gap of 250 feet still remained for navigation, a vessel inward bound for Alloa collided in the early hours of the morning with the end of the staging. Whilst no person was hurt, considerable damage was done to the staging and to the 5-ton derrick crane being used for its erection. The result of this was to cause a delay of practically 3 months, and this unfortunate and avoidable accident had its repercussions throughout the remainder of the contract by throwing this section of the southern portion of the bridge behind programme.

As finally completed, the staging extended from both shores to piers 11 and 12 respectively, a gap of 150 feet being left for navigation between these piers. For the guidance of shipping this staging was lit at close intervals from sunset to sunrise by electric lamps, the navigation opening being defined by special lamp-clusters.

Permanent Timber Jetty at Main Swing-Span Pier.

This timber jetty measures 470 feet long overall by 50 feet wide. Its purpose is to protect the swing-span structure when in the "river" position from damage by a vessel passing through either opening.

With this in view the round-nosed ends are substantially braced

and strutted to take the full impact of a vessel. The structure consists of four rows of piles 14 inches by 14 inches by 50 feet long driven about 10 feet into the sand and gravel of the river bed. The piles are at 10-foot 1½-inch centres longitudinally. A deck of 9-inch by 3½-inch timbers on 12-inch by 6-inch deck beams is laid at both sides of the jetty with an open well in the centre. Bollards are provided at intervals, although it is hoped that no occasion will arise for their use. Smaller but somewhat similar round-nosed protection fenders are adopted at the side piers Nos. 10 and 12 of the swing-span. In the case of the piles at pier No. 10 the depth of soft material overlying the rock through which the timber piles were driven was 1 foot 6 inches only. To ensure that no movement of the piles took place concrete blocks were cast around the piles at the river-bed level.

The timber used throughout on these permanent structures was creosoted Canadian Douglas fir, this being adopted in place of Australian turpentine timber which was the alternative Empire timber considered. The position of the bridge on the Forth is sufficiently far upstream to ensure a good admixture of fresh river-water which discourages the existence of either *teredo* or *limnoria*, although the latter is particularly active at the mouth of the estuary. Despite this it was decided to ensure as thorough a penetration of the creosote as possible into this timber, having regard to its well-known resistance to the absorption of creosote, and for this purpose all the timbers were incised in Canada before despatch. The incisions were ¾ inch deep spaced 1 inch apart transversely with a distance of 2 inches between successive rows. Each row of incisions is staggered by ¼ inch so that a space of 8 inches occurs between incisions on the same longitudinal line.

The timbers were unloaded from the vessel at Grangemouth docks about 3 miles downstream, where they were creosoted. The specification adopted involved their being in a preliminary vacuum equivalent of 20 inches of mercury for 30 minutes, and being then "soaked" for 6 hours in creosote of a temperature of from 100° F. to 160° F, followed by a gradual building-up of pressure during a period of not less than 2 hours to 180 lbs. per square inch maximum, this pressure being maintained thereafter for a minimum period of 2 hours.

The specification was rigidly adhered to and carried out most conscientiously by the creosoting firm. Under these conditions the amount of creosote injected into the larger scantlings averaged 3·95 lbs. per cubic foot, as compared with the 4 lbs. specified. For the smaller sections the amount increased to 6·9 lbs. per cubic foot with incised timber against the 6 lbs. specified. For unincised

timber the amount was 3·38 lbs. only for the smaller sections. The depth of penetration of the creosote was of the order of $\frac{3}{4}$ inch. Whilst the effect of incising the timber was to increase the impregnation per square foot of surface by two or three times that of unincised timber, it is possible that even higher values per cubic foot would have been obtained if a smaller interval had occurred between incising and creosoting.

It was specified that the main jetty was to be constructed in time to permit its being used as a staging for the erection of the swing span in the open "river" position. The jetty-construction was accordingly put in hand as soon as the temporary staging was extended out to this part of the river. The jetty was completed by the autumn of 1935. A fire-hydrant is installed on the jetty to enable any outbreak of fire to be dealt with rapidly.

Bridge Piers.

Commencing at the Kincardine end of the bridge, on the land section there are an abutment and two solid piers Nos. 1 and 2. The shore at this point consists of an area reclaimed when the railway was constructed, and was unsuitable to found on. The bores showed rock 15 feet down and accordingly the foundations were secured by driving 16-inch by 16-inch piles 20 feet long into the rock to refusal. Piles driven to a batter of 1 in 3 were introduced at the corners, etc., to provide greater stability. All the piles were driven by the adjustable piling-frame already described, which was later taken over to the south side for erection on the travelling gantry.

The north abutment is a mass-concrete structure supported on thirty-six piles.

Piers No. 1 and 2 are rectangular piers 36 feet by 6 feet at the top, each carried on twenty piles. The sides of the piers are formed to a batter of 1 in 30. As this section of the bridge is on a curve of 520 feet radius, superelevation had to be introduced on the roadway to comply with the requirement of the Ministry of Transport. The girder-seatings are, therefore, taken up in steps of 2·4 inches to suit the superelevation of 16 inches provided on the roadway, which was the minimum permissible.

Pier No. 3, which marks the junction of the 62-foot 6-inch spans with the 100-foot spans, was arranged to carry a large pilaster 9 feet wide and 14 feet high above road-level as an architectural feature of the design (Fig. 5, Plate 1). The pier itself consists of two circular reinforced-concrete columns 13 feet in diameter at the top and with a batter of 1 in 30. The columns are tied together by a reinforced-

concrete portal-beam with a curved underside, on top of which the ends of the six 100-foot and 62-foot 6-inch girders are carried.

The substrata at the site of pier No. 3 consisted of about 14 feet of mud and sandy clay with 5 feet of blaes and fireclay overlying rock, the mud being exposed at low water. The construction of the pier was effected by sinking steel cylinders 15 feet 6 inches in diameter, consisting of $\frac{1}{4}$-inch steel shell-plate stiffened by 3-inch by 3-inch angles and bolted together in sections 3 feet 10 inches long. The cutting-edge on the lower section was formed by a stiffening-plate 9 inches by $\frac{1}{2}$ inch. The cylinders were pitched by crane within the timber guides provided and the sinking was carried out by grabbing. The soft material overlying the rock was easily removed and the cylinders were lowered on to the blaes. At low water it was possible without difficulty for men to excavate in the dry within these cylinders until a satisfactory foundation was obtained on the sandstone below. About 1 foot 6 inches of good-quality coal was passed through in this foundation.

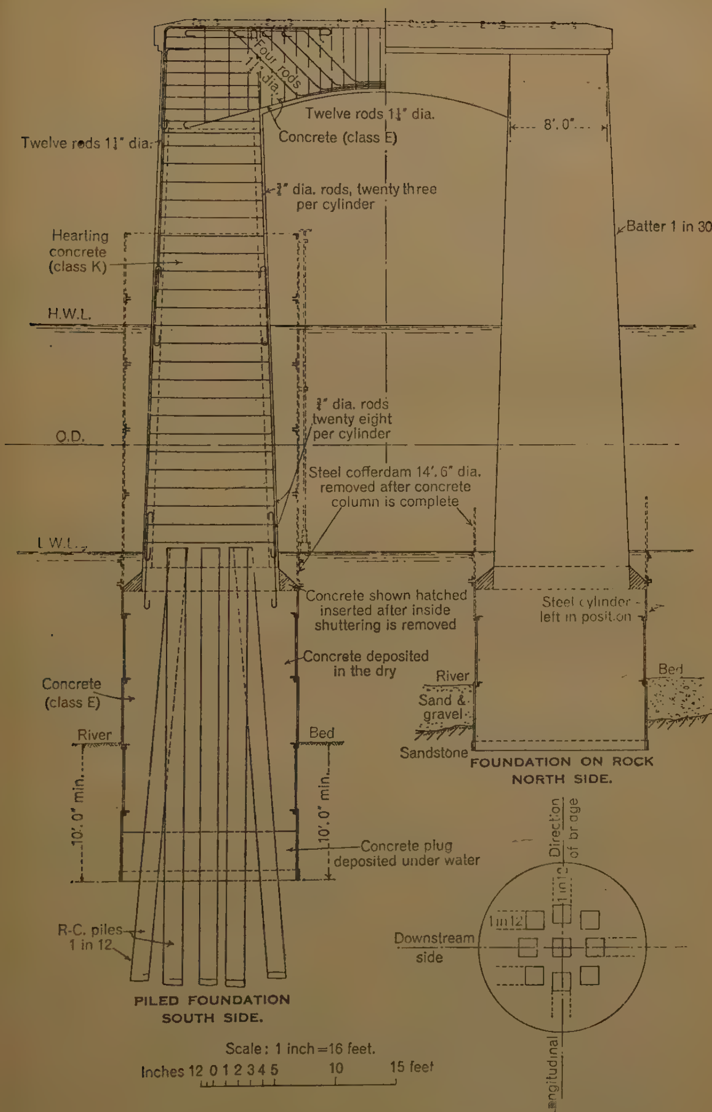
The base of each cylinder up to O.D. was filled with concrete (class "E") †, the steel shell which formed the pier being left in. Tapered steel shuttering was accurately set within the steel cylinder and the battered column constructed. The tapered columns in all cases consist of a 9-inch skin of concrete (class "F") within which are introduced $\frac{3}{4}$-inch diameter rods at 18-inch centres vertically and 40 per cylinder transversely, the steel being carried into the base to form a key. A 3-inch cover from the front surface of the concrete was given to this steel.

In addition the piers are heavily reinforced with $1\frac{1}{4}$-inch diameter rods to develop the portal effect and reduce the bending moment on the portal-beam (*Figs. 6*). The centre of the column consisted of filling concrete (class "K"), which was brought up at the same rate as the 9-inch outer-skin concrete by means of movable shuttering. Above the springing of the portal-beam the concrete adopted was of class E, the shuttering for this beam being also of steel. The contractors adopted steel shuttering throughout on the piers, and the high-class finish obtained on the concrete, with the advantage of being able to use two sets of shuttering only for the whole of the twelve standard piers, proved the wisdom of their choice.

Piers Nos. 4 and 5 were also constructed in the same manner by pumping the steel cylinder dry and excavating at low water within the cylinder through the material overlying the rock. The cylinders in the case of piers No. 4 to 9 are only 14 feet 6 inches in diameter, these being formed of steel plating $\frac{5}{16}$ inch thick with 4-inch by 3-

† See p. 713.

Figs. 6.



Typical 100-Foot Pier Details.

inch and 5-inch by 3-inch stiffening-angles. The cutting-edge consists of a 9-inch by $\frac{5}{8}$-inch plate. Sufficient steel cylinders were provided to allow of two sets of piers being proceeded with simultaneously, these cylinders being carried well above high water to form a cofferdam.

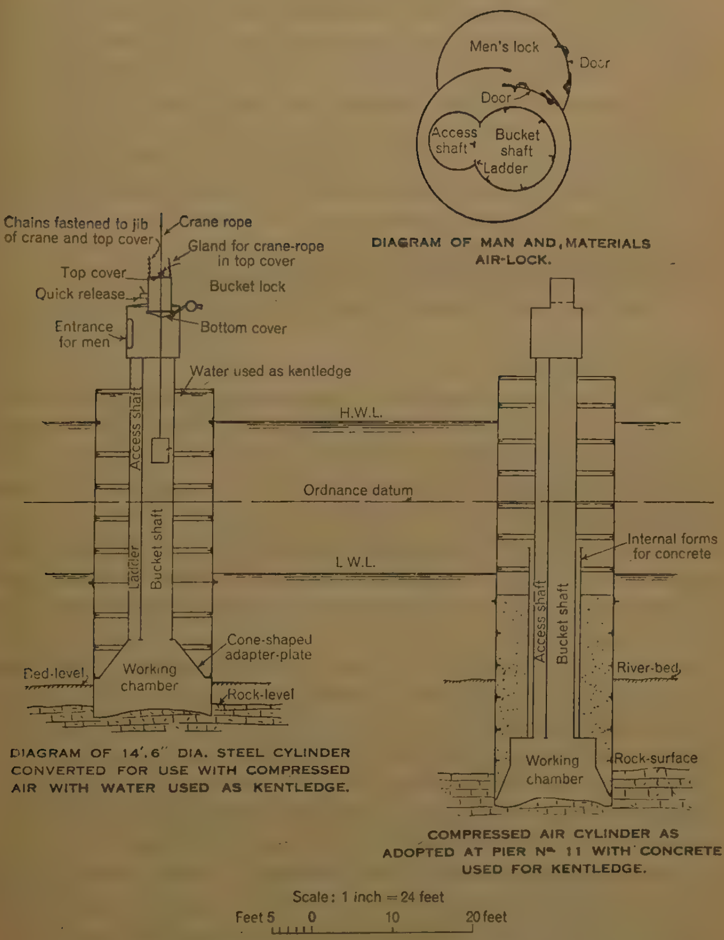
Each pier for the 100-foot spans consists of twin reinforced-concrete circular tapered columns as already described. The diameter of the column at the springing of the portal beam is 8 feet 4 inches in every standard pier with a taper of 1 in 30, so that it is clear that the same shuttering could be adopted in every case with making up pieces at the foot to suit the varying girder- and base-levels. In all cases the tapered column terminates in the 14-foot 6-inch base of the pier at a level of 3 feet below low water, the steel cylinder below this being abandoned. The bolts connecting the sections above the portion filled with concrete were loosened by a diver at low water, and the sections above this level removed and used again on the other piers. The junction between the 14-foot 6-inch base portion and the tapered column was chamfered off at 45 degrees with concrete (*Figs. 6*).

The foundations for pier No. 4 with a depth of 8 feet of mud and sandy clay overlying the sandstone were excavated in the dry and concreted without difficulty at low-water periods. At pier No. 5 the increasing depth of water experienced and the reduced cover of only about 3 feet of soft material overlying the rock introduced difficulties in sealing the base. Several "blows" occurred, necessitating the services of a diver to seal the cylinder by concrete in bags. It was possible, however, ultimately to excavate into the rock at selected tides and to obtain a satisfactory foundation.

Pier No. 6 was the last to be attempted with the open-cofferdam construction. A cover of 6 feet of soft material over the rock made the prospects of obtaining a seal at low water more hopeful, and some excavation was actually carried out. A sudden "blow" occurred during the excavating of the downstream cylinder, however, and it was decided thereafter not to endanger the workmen engaged on this excavation work. Arrangements had already been made for the supply of compressed-air equipment and this was assembled without delay, the compressor being installed on the shore at Kincardine with the air-supply pipes led out on the temporary structure. It was decided to revise the order of construction of the piers and to leave pier No. 6 in the meantime, commencing with the farthest out of the standard piers (No. 9) and working back towards No. 6. This was necessary because as soon as the portal-beam was set in position the upstream crane-track was no longer available to the travelling crane owing to lack of headroom.

The 14-foot 6-inch diameter cylinders already mentioned were adapted to take cone-shaped steel plating which formed the roof of the working chamber (*Figs.* 7). From this the access-shaft was led to the lock at the top. The shaft provided access both for men and

Figs. 7.



for materials, whilst the lock had a vertical opening for materials and a horizontal access for the men.

Water was pumped into the annular space between the shaft and the cylinder to provide sufficient kentledge to prevent uplift. Generally the water-level was kept about 8 feet above the tide-level

to give about 16 tons margin against movement and to assist in sinking the cylinder to its final position. The adoption of compressed air enabled the excavation and concreting of the foundations of the remainder of the pier to be accelerated, as the work could be proceeded with continuously independent of the tidal conditions.

After the rock had been excavated to a depth of about 3 feet to enable a good key to be obtained, the foundation concrete was placed with the cylinder still under compressed air. Pipes were passed under the cutting-edge of the cylinder, which was in all cases lowered down to the bottom of the excavation, and were brought up inside to a height above the level of the concrete plug. This was to prevent movement of air or water through the setting concrete. After the concrete (class "E") had set, the pipes were sealed by caps and the pressure released thus enabling the cone-plate, access-shaft and air-lock to be removed to the next cylinder. To enable the cone-plates to be removed the water ballast was substituted by about 200 tons of solid kentledge and the remainder of the work above the plug proceeded with as an open well until the weight of deposited concrete permitted the removal of the kentledge.

The piers from No. 9 back to No. 6 were constructed in this manner without incident, and in every case satisfactory foundations in grey sandstone were obtained. About five men were engaged in the working chambers, operating in shifts of 10½ hours. The maximum pressure experienced in this part of the scheme was 18 lbs. per square inch at pier No. 8. All the men engaged under compressed air were medically examined and passed as fit. For this reason and on account of the moderate air-pressures experienced no cases of illness occurred on the work. The rate of construction of the piers was about one every month. Full particulars of the foundation-levels and construction dates for each pier are given in Appendix I.

It will be convenient to deal here with the corresponding piers, Nos. 13 to 18, on the south side of the bridge, although these were constructed after the piled viaduct and 50-foot spans yet to be described. These piers are carried on groups of eighteen piles 18 inches by 18 inches, nine of which were driven in each of the two cylinders comprising a pier. The arrangement of the piles is shown in *Figs. 6*, and consists of four vertical piles and five raking piles driven to a batter of 1 in 12. With this arrangement stability is obtained in both longitudinal and transverse directions.

The piles were driven by means of the overhead gantry travelling on the line of temporary staging, with its piling-frame on top capable of being moved to command each foundation and also rotated and inclined to drive the batter piles. The piling carriage was 90 feet long and the hammer adopted was a single-acting steam

hammer weighing 5 tons. The set specified was not to exceed 1 inch to six blows with a drop of 3 feet 9 inches for 55-foot piles, and 4 feet 3 inches for 65-foot piles. This set was obtained without difficulty. The total weight to be carried by each pier, including its dead load, was about 1,250 tons divided over eighteen piles.

To check that the piles would, as had been assumed, draw up in the gravel bed disclosed by the borings, the travelling gantry with its piling-frame was run ahead and test-piles were driven at pier No. 16. From the results obtained the construction of piles of suitable length was put in hand.

The piles, as already stated, were 18 inches by 18 inches and generally 55 feet and 65 feet long. The reinforcement consisted of four rods each $1\frac{1}{2}$ inch in diameter with $\frac{3}{8}$-inch diameter links at 9 inches centres, this spacing being reduced to 4 inches and 6 inches at the bottom, and 2 inches, 4 inches and 6 inches at the top. Lifting holes were provided at the fifth-points. The concrete used was class "F" mixture, rapid-hardening cement being adopted.

For handling the piles, which weighed about 9 tons, into position on the piling-frame, the procedure adopted in all cases on the south side was to travel the piles out on bogies by means of a travelling crane. A 10-ton Scotch derrick-crane travelling on bogies on the two tracks ahead of the gantry lifted the piles and set them in position at the frame. The piles drove very easily through the upper layers of material until the hard stratum of gravel was reached where they generally drew up after penetrating a few feet into the material. In such cases where they passed through the gravel bed a general stiffening-up took place until the final set was obtained. The pile-heads were protected by paper or sacking in a helmet of cast steel, the top of which carried a timber dolly.

The sequence of operations for the 100-foot-span piers on the south side was to sink the 14-foot 6-inch diameter cylinder into the bed of the river for about 10 feet, in order to be below the effect of possible future scour. Except in certain cases where boulders interfered with the cutting-edge and had to be removed by a diver, no difficulty was experienced in sinking these cylinders to the required depth by grab. When sunk to its final position the nine piles were driven within the cylinder as already described.

It was necessary in certain piers near mid-river to lower the water-level within the cylinder to enable the pile-driving to be carried out "in the dry." The seal produced by the 10 feet of soft material was sufficient to enable this operation to be successfully carried out, but care was taken, however, to reduce to a minimum the time required for the operation of driving the pile and disconnecting the guides. A sluice fitted below low-water level in the side of the

cylinder permitted the upward pressure on the bottom to be rapidly released by balancing the water-level. The disconnexion of the guides required the water-level to be at least 5 feet below the head of the pile. The maximum head of water that the cylinders were subjected to during the driving of the piles on the south side was 15 feet.

After the pile-driving was completed in a cylinder, concrete (class "E") was deposited through water by means of a special bucket with a hinged base which, after reaching the bottom, was opened to allow the concrete to escape. By this means a concrete plug from 4 feet to 5 feet thick was formed, and as soon as this had set the well was pumped dry.

From *Figs. 6* (p. 701) it will be seen that generally about 18 feet of the piles projected through the concrete plug. These heads were picked over to roughen the surface, the mud and slurry in the bottom were removed, and the concreting of the base was completed up to the required level. The remaining lengths of pile-heads (if any still projected above the base-level) were cut to leave about 3 feet to be built into the tapering portion of the pier which was completed by means of steel shuttering in the manner described already.

Pier No. 19, which marked the junction between the 100-foot and 50-foot spans, is of similar dimensions to pier No. 3. The total number of piles in the base of this pier is eighteen as in the case of the other piers.

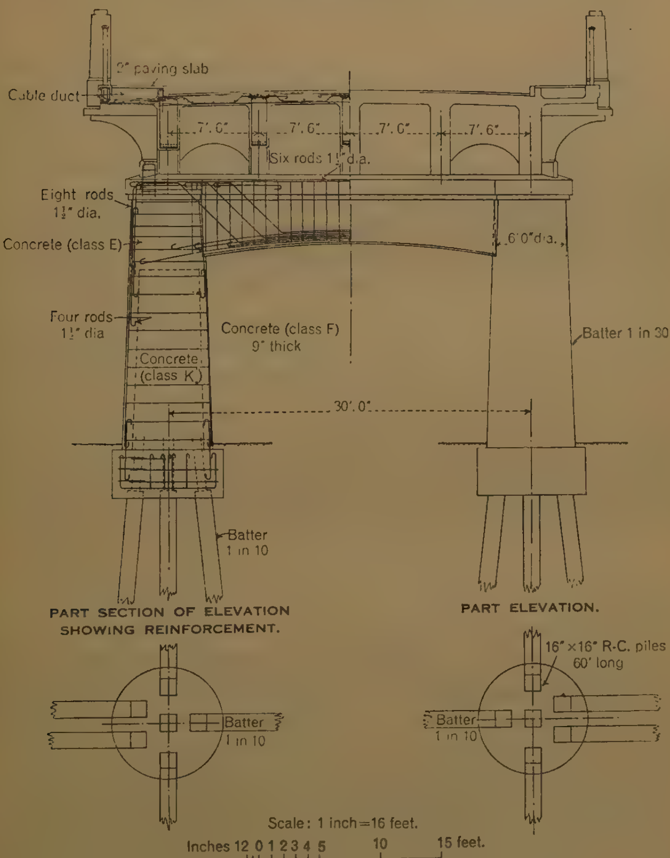
Piers for 50-foot Reinforced-Concrete Spans.

The piers carrying the 50-foot reinforced-concrete spans are circular tapered columns somewhat similar to those of the 100-foot span-piers. The width at the top is 6 feet in this case with a batter of 1 in 30. Each base of the piers is supported by a group of six reinforced-concrete piles as shown in *Figs. 8*, five being driven to a batter of 1 in 10 and the centre pile being vertical. This grouping was arranged to ensure adequate stability in both directions. The piles adopted were 16 inches by 16 inches, from 55 to 65 feet long, the lengths being determined from preliminary test piles driven before the main group of piles were constructed. The piles are reinforced with four 1¼-inch diameter rods, but otherwise they are generally similar to the 18-inch piles already described. The piles were driven by a 5-ton steam hammer with a 3-foot 3-inch drop, the specified set being not more than 1 inch for six blows. During the driving of the piles at piers No. 20, 21, 25 and 26, some soft patches in the gravel were experienced. To obtain additional support two extra piles were driven in each cylinder. It was decided to leave for 1 or 2

months certain of the piles which had not reached their specified set of 1 inch to six blows, in order to determine whether they would seize up. The re-driving tests made after this period showed that this had taken place, the set obtained being well within what was required.

The saltings on which the piers are located are covered only at

Figs. 8.



TYPICAL 50-FOOT PIER DETAILS.

spring tides, and the reinforced-concrete block tieing the heads of the piles together and forming the base-slab of the pier was constructed within timber sheeting and excavated and concreted in the dry. The tapered columns for the 50-foot-span piers were formed within steel shuttering in a similar manner to the 100-foot-span piers, with

a facework of class "F" concrete 9 inches thick reinforced with $\frac{3}{4}$-inch-diameter rods in both directions, the hearting of the pier being of class "K" concrete.

A peculiar feature of all the piling executed on the south side was that the level at which the point of the piles drew up with the required set in the case of the upstream piles for piers Nos. 13–28 was in practically every instance lower than that of the downstream piles by amounts varying from 5 to 15 feet.

Piled Viaduct.

This portion of the bridge is constructed of 18-inch by 18-inch piles, generally 65 feet long, driven about 3 feet into the stratum of gravel 50 feet below ground-level. The piles are at 10-foot centres longitudinally and at 15-foot centres transversely, and support a 10-inch reinforced-concrete slab forming the carriageway, with an 8-inch slab cantilevered out to form the footpaths. The viaduct is subdivided into sections 52 feet 6 inches long for dealing with expansion. At these places the crossbeam is widened and five piles instead of three are provided for greater stiffness. The free end of one slab rests on the half width of this widened beam, and the sliding joints are painted with bitumen and operate very satisfactorily in practice. The end of the viaduct where it connects with the south approach road has the end group of piles braced to the adjacent rows of piles by inclined and horizontal struts buried below the ground-surface to take up any end thrust from the embankment.

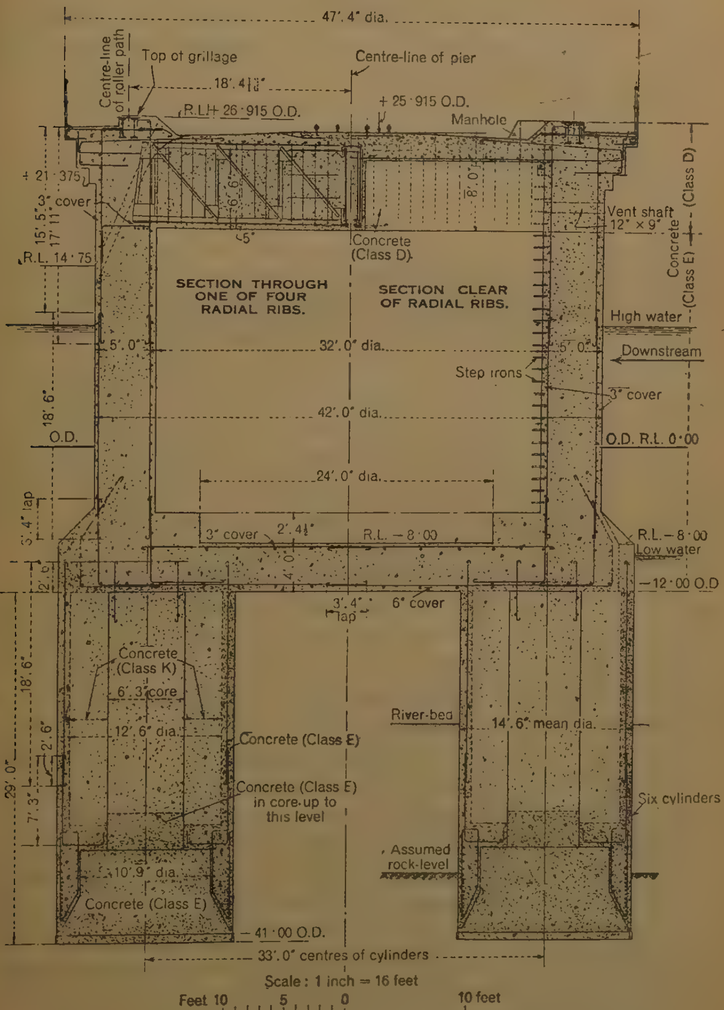
Swing-span.

Centre Pier.—As designed, pier No. 11, which carries the 1,600-ton moving weight of the swing-span, consisted of a hollow circular pier 42 feet in diameter outside with walls 5 feet thick. The concrete foundation covering the whole of the base area was to be taken down to solid rock at about − 41 O.D. For this purpose the whole of the work was to be carried out in the dry within a circular steel-sheet cofferdam 50 feet in diameter. The cofferdam, consisting of No. 2 Larssen steel sheet-piling and securely braced with five steel circular walings, 2 feet 2 inches deep, was driven through the 12 feet of overlying sand and gravel to rock-level.

Although boulders were experienced at places, preventing a complete seal with the rock, it was anticipated that a reasonably tight cofferdam had been provided. De-watering by pumps was put in hand during December, 1934, without much success. Additional pumping plant was brought forward until at one time three pumps with a total capacity of 6,500 gallons per minute were in action.

Several minor " blows " occurred and on the 9th January, 1935, when the bottom of the river bed had actually been exposed in places inside the cofferdam, a violent " blow " occurred at high water

Fig. 9.



Main Pier Carrying Swing-Span.

under the feet of the piles, dragging in about 15 or 20 feet of the sheeting up to the point where they were held by the lowest waling, and flooding the cofferdam in a few minutes.

It was evident that complete reconsideration would have to be given to the sealing of this cofferdam, or alternatively an entirely new design would have to be evolved. The latter offered the greater prospects of accelerated progress, and accordingly the modified design shown in *Fig. 9* was adopted. This, it will be seen, consists of six cylinders 14 feet 6 inches in diameter, equally spaced out on a diameter of 33 feet and carried up to just below low water, where they are tied together by a solid and heavily-reinforced concrete slab 4 feet deep. Above this the 42-foot diameter cylinder is constructed as in the original design.

The steel shuttering for the 14-foot 6-inch diameter cylinders was already available at the site, being surplus from the 100-foot-span piers then completed, and the proposal was to sink the cylinders for the centre pier under compressed air into solid rock, in a similar manner to the 100-foot-span piers. In order to avoid the necessity of bringing forward kentledge for the cylinders it was decided to use the permanent concrete to give the required load against uplift. The steel cylinders were supported from three pairs of jacks, and perforated link-plates with large-diameter pins were provided to carry the load while the jacks were released and reset. The whole load was carried on temporary staging, and as concreting proceeded within the steel cylinders they were gradually lowered into the water to obtain a reduction by water-displacement of the total load on the jacks.

By this means the whole of the steel cylinder with its encasing concrete and the internal air-lock shaft in position was lowered on to the river-bed, when excavation under air was put in hand until a sufficiently stable support had been obtained to permit the jacks supporting the load to be dispensed with. Excellent foundations on sandstone of particular toughness were obtained for all six cylinders at depths of about — 38 O.D., these being keyed about 3 or 4 feet into the rock. The maximum air-pressure experienced was 21 lbs. per square inch. The working chamber and central shaft was then concreted up to the level of the underside of the slab at — 12 O.D. A few lengths of steel sheet-piling were driven to seal the gap between the cylinders, and the whole of the inner space was filled up with gravel to this level to simplify the construction of the slab. The base of the slab is 3 feet below low water but the cofferdam was sufficiently tight to withstand a few feet difference in head, and the slab was thus constructed in the dry without difficulty.

The top of the pier is supported by two steel lattice-girders 6 feet 6 inches deep arranged at right angles. At their junctions the eight $2\frac{1}{4}$-inch diameter U-bolts to carry the centre pivot are located. These steel girders are totally encased in concrete. The roof of the

pier consists of a reinforced-concrete slab from 1 foot 7 inches to 2 feet 3 inches thick sloped from the centre to the sides for drainage. A manhole provides access to the inside of the pier and gratings are also provided to ventilate the interior. The external circular shuttering of the pier, together with the corbelled top courses, were of welded steel throughout, and an excellent finish to the concrete was obtained by this means.

The total weight of piers and swing-span coming on the six cylinders is estimated at 4,200 tons, so that the rock has to carry only the very moderate pressure of $4\frac{1}{4}$ tons per square foot. The construction of the pier occupied 8 months from the date when the first of the six cylinder-foundations was approved for concreting.

North End Pier.—Pier No. 10, which carries the end of the swing-span when in the closed position, as well as the large ornamental portal with the safety gates, etc., had been designed to be constructed in the dry within a steel cofferdam as in the case of the centre pier. It was decided in the light of the experience gained on pier No. 11 to modify the design and to incorporate two large cylinders 21 feet in diameter for this pier, to be sunk under compressed air to the rock. Part of the steel sheet-piling had already been driven at this pier, and this was completed, together with the steel walings.

In view of the large size of cylinder, four sets of jacks were required to support it, but otherwise the methods adopted were similar to those already described for pier No. 11, and excellent foundations were obtained on tough grey sandstone at about — 26 O.D.

A slab 60 feet by 22 feet by 4 feet 6 inches thick with substantial reinforcement joins the two cylinders together, and carries the hollow pier and its superstructure. The base of the slab is 1 foot below low water, and by means of the cofferdam the water-level was kept down sufficiently to permit the base of this slab to be shuttered and concreted in the dry. Steel shuttering was used and connected to the steel shells of the 21-foot-diameter cylinders. To simplify the operations within the cofferdam the whole of the reinforcement cage for the base-slab was assembled, wired together on shore and lowered by crane into position inside the cofferdam.

The pier above the slab consists of a hollow rectangular section with sides tapering at 1 in 30 and with semicircular ends, the walls being 5 feet thick at the base and 3 feet 6 inches thick at the top. The internal area is subdivided by three cross-walls 3 feet 6 inches thick. The pier has a reinforced-concrete roof 4 feet thick on which are carried the ends of the 100-foot girders and the supports for the swing-span. Steel shuttering was used for the whole of the

external facework with very satisfactory results. Access-manholes and ventilators are provided for this pier.

The ornamental portal across the roadway is also supported by this pier, which thus extends from −26 O.D. to +74·5 O.D. The load transferred to the rock from this pier is of the order of 5·8 tons per square foot. The construction of this pier and its superstructure took place between August, 1935, and August, 1936, the portion up to the girder-level being completed by April, 1936.

South End Pier.—The foundations of pier No. 12 as designed were also intended to be constructed in the dry. Further consideration of the broken and unreliable nature of the substrata at this pier due to the fault, as well as of the failure of the attempts to de-water the cofferdam at pier No. 11, indicated the prudence of revising the design for the foundation.

It was ultimately decided to support the whole load from this pier on a piled foundation constructed within the cofferdam. The load to be carried necessitated the provision of seventy-six piles 18 inches by 18 inches by 40 feet long driven in two rows. The forty-two piles in the outer row are driven to a batter of 1 in 10 to give greater stability against side thrust, while the inner row is vertical. The steel cofferdam was first formed, the sheet-piles being driven without difficulty to a level of 15 feet below the river-bed. The whole of the area within the cofferdam was then excavated by grabbing to a level of 10 feet below the river-bed and the reinforced-concrete piles then driven by a 5-ton hammer from a special piling-frame erected within the cofferdam. Each pile was driven practically to refusal on to the broken metals at a level of about −47 O.D., or 20 feet below river-bed level. The length of the piles was such as to permit all driving to be done above low-water level.

A plug of concrete 3 feet 6 inches thick was deposited through water to seal the whole of the area within the cofferdam and when this had set the cofferdam was pumped out without difficulty and the remainder of the construction proceeded with in the dry. A base of concrete 65 feet by 27 feet by 7 feet 6 inches thick, reinforced with 1-inch diameter steel rods, was formed on top of the concrete plug. The projecting piles were roughened and certain of the heads cut off and their steel exposed in order to be incorporated into this base. Above this the construction of the pier within steel external shuttering was similar to that of pier No. 10, the same shuttering being adopted. The construction of this pier to girder-level occupied the period from March to June, 1936, the superstructure being completed by September.

The total number of reinforced concrete piles driven on the whole scheme was four hundred and eighty-four.

Concrete Aggregates and Mixtures.

The cement used throughout the whole of the scheme was English Portland cement of ordinary-setting quality, other than for the piles, where rapid-hardening cement was used. At the commencement of the work the contractor used crushed whinstone from local quarries for aggregate, with sand imported from the Earn river in Perthshire. Further investigation indicated the desirability of opening up a special pit for sand close to the site, and ultimately a sand and gravel quarry near Denny in Stirlingshire, about 9 miles from the bridge, was selected.

Tests showed that the sand and gravel from this quarry gave excellent results, and crushing and washing plant were installed to provide the whole of the materials for the concrete. The sand, although dark in colour, gave very good results, figures for a test at 3 days compared with those for briquettes made with standard sand and ordinary cement (3 to 1) being as follows :—

	Strength.
Standard sand : Leighton Buzzard	460 lbs. per square inch
Sand from Denny	458 ,, ,, ,, ,,
Sand from Ham river	447 ,, ,, ,, ,,

During the progress of the works about three hundred compression tests were made on 6-inch cubes. The following Table gives the concrete mixtures adopted, the minimum compressive strengths specified, and typical results obtained :—

Concrete : class.	Cement : cwts.	Sand : cubic feet.	Aggregate : cubic feet.	Specified strength at 7 days : lbs. per square inch.	Average strength obtained : lbs. per square inch.
"K"	$2\frac{1}{2}$	12	20	—	—
"D"	4	12	20	1,600	2,100
"E"	6	12	20	2,000	3,150
"F"	8*	12	20	3,600	4,500

* Rapid-hardening cement.

The distribution of the various classes of concrete was generally as follows :—

Class "K": hearting of cylinders.
Class "D": beams and deck in piled viaduct and 50-foot spans. Concrete on roadway of steel spans.
Class "E": plugs and foundations under water. Walls of centre and end piers of swing-span, portals and pilasters.
Class "F": piles and 9-inch facing to 100-foot- and 50 foot-span piers. Pilasters and girder-blocks.

The size of the aggregate used was ¾ inch for all reinforced-concrete work and 2 inches for piers, etc.

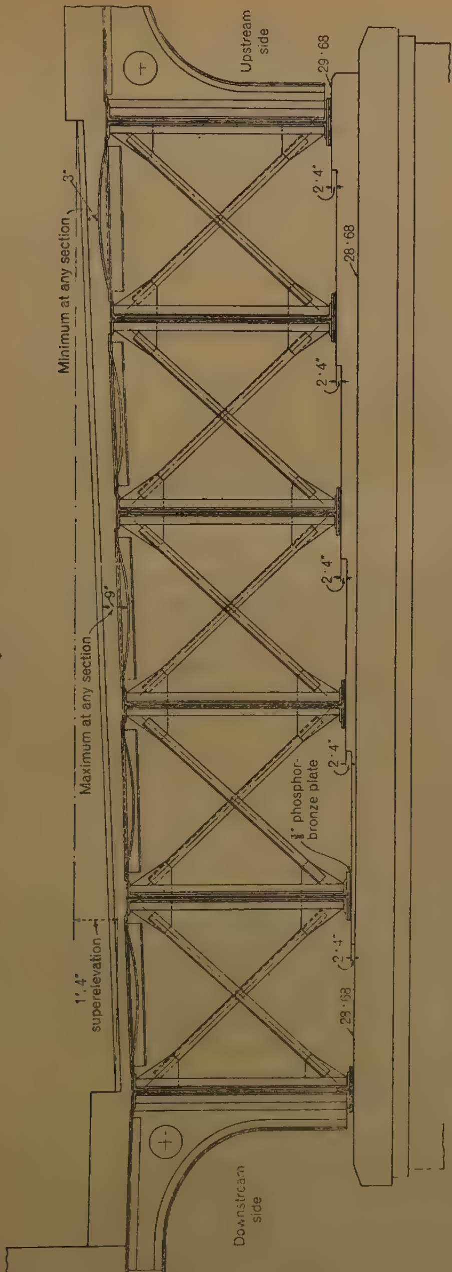
Steel Spans.

The superstructure of the bridge was designed to carry the latest live loads specified by the Ministry of Transport, which were 208 lbs. per square foot plus the knife-edge load of 2,700 lbs. per foot of width for the 100-foot spans. For the footpaths a live load of 84 lbs. per square foot was adopted in accordance with the values given in British Standard Specification No. 153. The general requirements for the structural steelwork and the permissible working stresses were adopted from that Specification.

The three 62-foot 6-inch spans carrying the bridge over the L.N.E. Railway on a curve of 520 feet radius were designed as continuous girders. The continuous form of construction was selected to permit of a curved outline of girder being adopted as in the case of the main 100-foot spans. For the 30-foot width of roadway six longitudinal girders at 6-foot centres were adopted. The footpaths are cantilevered out from the outer girders (*Fig. 10*).

It had been intended in the first instance to construct this curved portion of the bridge without superelevation, but at the request of the Ministry of Transport, 16 inches of superelevation was allowed for. Transitions, both on plan and in section, were provided, as well as a gradual widening of the roadway to 32·25 feet in the centre of the curve to give additional space for the increased width taken up by vehicles on a curve. This gradual variation in three directions presented a problem in dimensioning the detail drawings for the steel fabrication, which was only solved in the case of the cross-bracing after the three curved spans had been assembled in the makers' works at Darlington. The results obtained, however, are very satisfactory from the point of view of the road user and thoroughly justify the decision to provide superelevation at this

Fig. 10.



Cross-Section of 62-Foot 6-Inch Spans Looking Towards Pier No. 1.

place. To reduce the gradient on this part of the bridge connecting with the approach road the clearance over the railway was reduced to the minimum acceptable to the railway company.

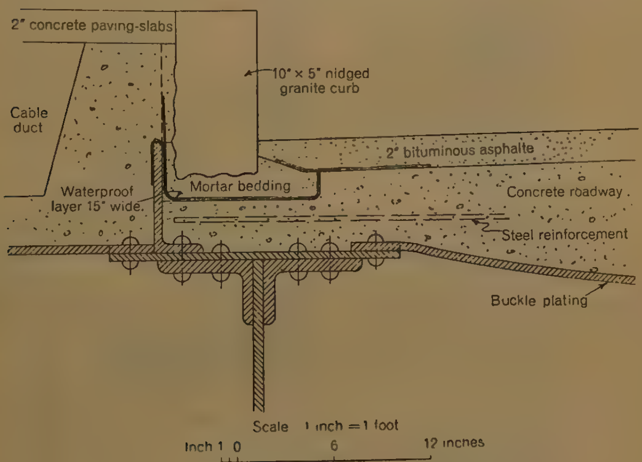
The girders consist generally of a $\frac{3}{8}$-inch web-plate $6\frac{3}{4}$ feet deep at the ends and $3\frac{3}{4}$ feet deep at mid-span. The flanges consist of two 6-inch by 4-inch by $\frac{3}{4}$-inch angles with 18-inch and 12-inch flange-plates. The bottom flange is formed to a curvature of 155 feet radius. Web-stiffeners consist of 5-inch by 3-inch angles at 3 feet 4 inches centres, with diagonal cross-bracing at 10 foot centres. The decking consists of $\frac{3}{8}$-inch buckled plates which, to suit the superelevation, are turned with their convex sides downwards. For the same reason the girder seatings are each set up by 2·4 inches across the width of the piers. The girders are fixed at piers Nos. 1 and 2, the ends at the north abutment and at pier No. 3 being free to slide.

The seven 100-foot spans on both sides of the river are designed as cantilevers, the 50-foot suspended girders occurring on the centre of the spans between piers Nos. 4 and 5, 6 and 7, and 8 and 9 on the north side, and between Nos. 13 and 14, 15 and 16, and 17 and 18 on the south side. This arrangement permits an economical adoption of the arched form of girder which is the feature of the design. Across the 30-foot width of roadway six longitudinal girders are spaced at 6-foot centres. These girders are constructed of $\frac{3}{8}$-inch web plate 9 feet deep at the supports and 5 feet at the centre over the angles. The flanges consists of two 6-inch by 6-inch by $\frac{5}{8}$-inch angles with an appropriate number of flange-plates 18 inches wide, the lower flange being formed to a curvature of 290 feet radius. The stiffeners occur at 5-foot centres and consist of 5-inch by 3-inch angles. Diagonal bracing-angles at 10-foot centres are fitted between the girders where the stiffeners occur. The deck of the roadway is formed of $\frac{3}{8}$-inch thick buckle-plate in sections 20 feet long and 5 feet wide with four buckles each measuring $4\frac{1}{2}$ feet by $4\frac{1}{2}$ feet by $3\frac{1}{2}$ inches, and being stiffened with a transverse channel at 5-foot centres To suit the camber of the road (1 in 45) the three inner buckles are turned with the convex face upwards, the side buckle next each curb being turned in the opposite direction.

The footpaths, 5 feet wide, are formed by curved cantilever brackets at 10-foot centres tied together at the outer edge with a 12-inch by $3\frac{1}{2}$-inch channel (Figs. 11, Plate 1). On top of the brackets is set a flat plate on which the concrete cable-duct is formed. The roadway itself is formed of concrete of $6\frac{1}{2}$ inches average thickness laid on top of the buckle-plates and reinforced with steel fabric The wearing surface consists of 2 inches of bituminous asphalt The curbs are of granite 10 inches by 5 inches in section, and to

minimize the risk of a vehicle mounting the pavement an 8-inch step is given by the curb. After laying the cables, water-pipe, etc., the footpath ducts were filled with sand and gravel and covered with 2-inch precast paving slabs. The concrete duct was painted with two coats of bituminous paint to improve its watertightness. For the same reason the vulnerable point for the penetration of water at the junction of the roadway and curb was made more secure by the insertion into the concrete of two layers of hessian canvas coated with bitumen (*Fig. 12*), so as to avoid the concrete drawing away and permitting the water to pass through to the buckle-plating.

Fig. 12.



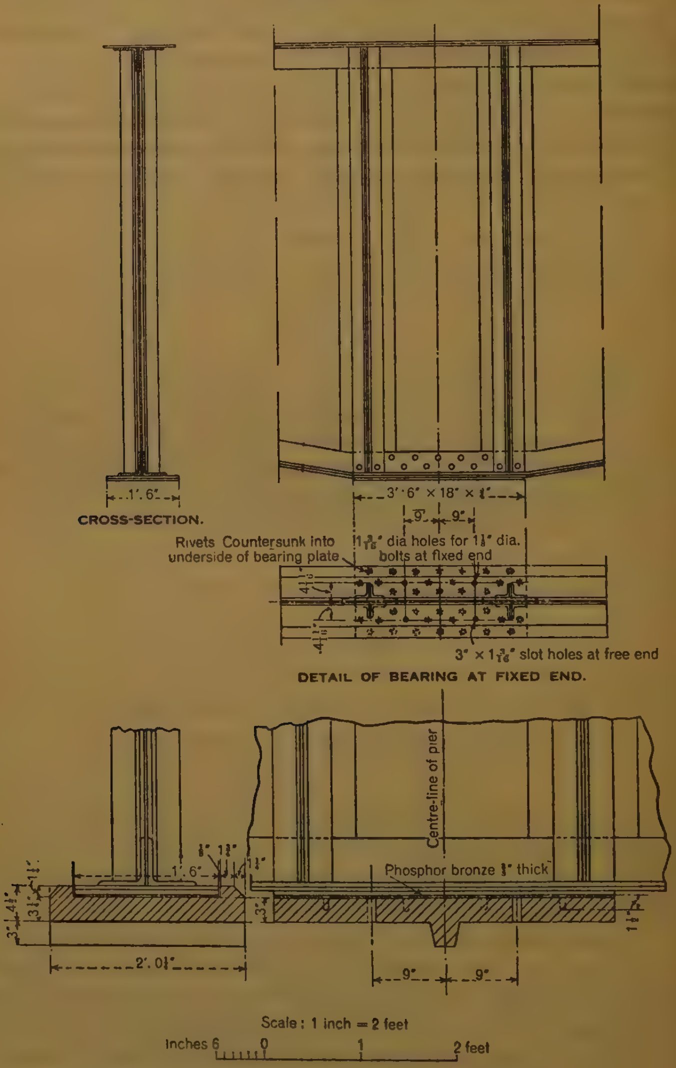
Waterproofing at Curb.

One of the reasons for the adoption of a light-coloured granite curb was to show up the edge of the road clearly to motorists.

The bearings of the 100-foot spans on the piers are alternately fixed and sliding, the fixed bearings being on piers Nos. 4, 6, 8 and 10 on the north side and on Nos. 12, 14, 16 and 18 on the south side. The sliding bearings consist of a substantial cast-iron bedplate 3 inches thick with a projecting tongue to key it into the concrete pier (*Figs. 13*, p. 718). A $\frac{3}{8}$-inch phosphor-bronze plate, 3 feet 6 inches by 18 inches, is securely fastened to the top of the bearing by countersunk screws. A $\frac{3}{4}$-inch thick steel plate of similar dimensions on the girder slides on this phosphor-bronze plate. Each girder is in every case held down by four $1\frac{1}{8}$-inch diameter bolts, slotted holes being provided at the sliding bearings as well as holes for grouting-up the

bedplate. These bearings have been found to work perfectly since their assembly.

Figs. 13.

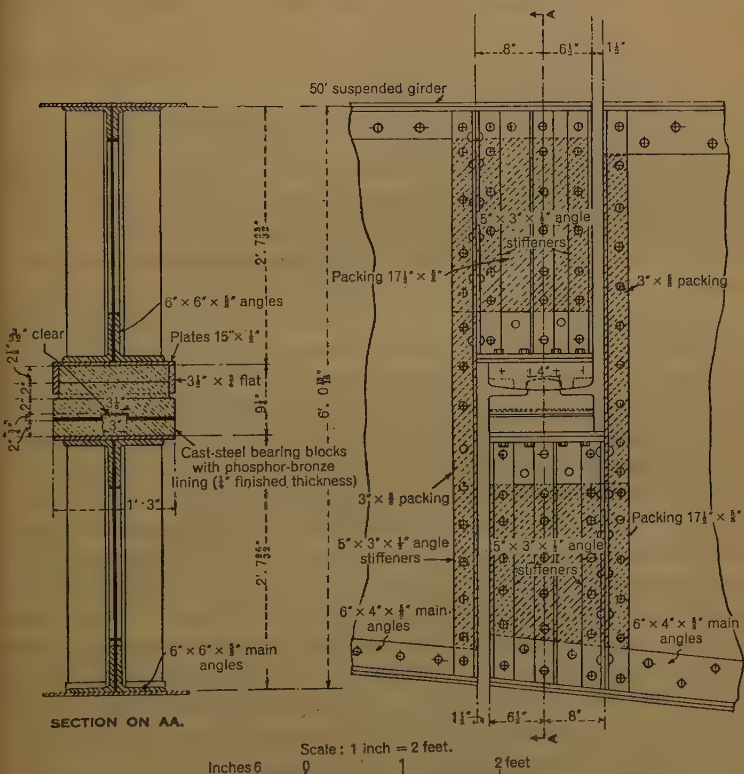


BEARINGS FOR 100-FOOT SPANS AT PIERS.

The 50-foot suspended spans rest on bearings fitted to a projecting end of the cantilever arms (*Figs. 14*). The cast-steel bearings, 15

inches by 13 inches by 3 inches thick at both ends of the suspended span, have a socket with a transverse axis which engages with a knuckle on a similar bearing fastened to the cantilever. By this means a rocking motion is permissible without allowing for any longitudinal movement in the case of the fixed end. At the other end of the suspended span the bearing is seated on a phosphor-

Figs. 14.



Bearing for Suspended Girder.

(100-Foot Spans.)

bronze lining, ¼ inch thick, which permits expansion and contraction to take place freely as well as the rocking motion.

Erection.

The adoption of cantilever construction facilitated the erection of the steel girders. On the north side the only track available at

the time was that immediately downstream of the piers, and this was used for the haulage of the whole of the steel to be erected on the scheme, amounting to about 4,000 tons. The steel was delivered by the railway company to Kincardine station alongside the bridge.

The 100-foot girders were delivered to the site in three sections :—

(i) The 25-foot cantilever end and a corresponding portion of the continuous girder, thus forming a length of girder weighing about 10 tons.

(ii) The 50-foot central section of the continuous girder, weighing about $8\frac{1}{2}$ tons.

(iii) The 50-foot suspended girder, weighing about 8 tons.

Two 5-ton locomotive cranes picked up girder section (i) and carried it out to the appropriate pier, where it was set on the downstream side of the pier, temporary trestle-supports to carry the ends having been erected from the staging. A similar section was thereafter erected on the adjacent pier and steadied in the same manner. Section (ii) was thereupon set by the cranes into position between the projecting ends of the continuous girder and bolted. This completed a continuous girder with its cantilever ends. Another girder alongside was assembled in a similar manner and the two girders securely braced together. The two girders, weighing together about 55 tons, were rolled over by jacks into position at the upstream side of the pier on steel balls operating in a V-shaped track which was set at a level to clear the projecting heads of the anchor-bolts. After being set in proper alignment the girders were lowered on to their seatings by jacks. By this means the six longitudinal girders on a pair of piers were assembled in position. Erection was commenced on the next pair of piers, and when these girders also were set in position by the same methods there only remained the gap to be filled by the suspended girders. These girders were also assembled on the downstream side and pulled over one at a time on a skid-rail fastened to the top of the cantilever end.

The steel for the 100-foot spans on the south side was taken out in wagons to the jetty at pier No. 12, where it was set by crane between timber guides fixed on two pontoons securely braced together. The pontoons were towed across at slack water by motor-boat to the south side staging, where they were lifted by two 5-ton cranes and set in position in a similar manner. The erection and assembly of all the steelwork was carried through with expedition and without the slightest difficulty, and reflected great credit on the excellence of the workmanship. Before the dispatch of the steel to the site three complete 100-foot spans, two girders wide, were

erected at the works at Darlington, and were found to fit together perfectly.

The riveting of the steelwork on the site was done by pneumatic hammers, compressors being installed for this purpose on either side of the river. The total number of field rivets driven was 150,000. For the riveting of the swing-span five gangs of riveters were in action at one time, the maximum number of rivets driven in a day being about 3,000.

50-Foot Spans.

Nine reinforced-concrete spans carry the road over the saltings on the south shore, between piers Nos. 19 and 28. The 30-foot width of roadway is supported by five reinforced-concrete beams with a curved underside to harmonize with the appearance of the steel spans. The transverse spacing of the beams is 7 feet 6 inches, the three inner beams being 20 inches wide, 5 feet 8 inches deep at the support and 3 feet 8 inches at mid-span (*Figs. 8*, p. 707). Curved cantilever brackets at 10-foot centres carry the footpath with its cable-duct in a similar manner to the steel spans, as well as a concrete pilaster. The outer beam, 22 inches wide, is stiffened against the cantilever effect by a curved jack-arch connecting the outer girder with the inner adjacent girder. The deck-slab, 10 inches thick, is monolithic with the beams.

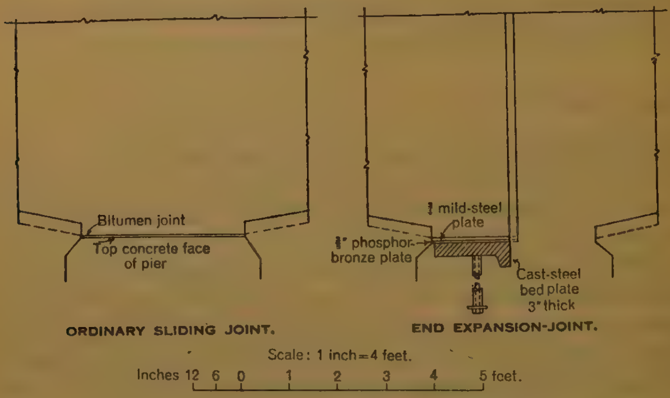
It was obvious that any attempt to provide support for the shuttering from staging set in the saltings would be doomed to failure, and accordingly the contractors designed special steel shuttering to span from pier to pier. This shuttering consisted of steel girders forming the sides of the shuttering for the beams. Shuttering sufficient for two complete spans was provided to accelerate construction. The necessity of providing openings to take the cantilever brackets, jack-arching, etc., involved considerable holing of the steel shuttering and reduced the effective strength of the girder so formed.

The method of construction ultimately adopted was to pour the concrete up to the level of the underside of the deck-slab only, to facilitate the removal of the shuttering. Closing spaces in the beams were left over the pier to guard against contraction cracks during the setting of the beams. The steel forms were removed at the end of 7 days and timber shuttering for the deck-slab was erected supported from walings bolted to the sides of the beams through holes left for that purpose. The gap in the main beam at the pier was filled at the same time as the deck-concrete was deposited. The nine spans are divided into groups of three to allow for expansion.

The fixed ends of the beams are made monolithic with the piers

by means of four rods, $1\frac{3}{8}$ inch in diameter; the sliding joints of the intermediate supports at the piers are bitumen-coated, whilst at the free end provision has been made for expansion (at piers Nos. 19, 22, 25, and 28) by incorporating into the underside of the beams at

Figs. 15.



PROVISION FOR EXPANSION ON 50-FOOT REINFORCED-CONCRETE SPANS.

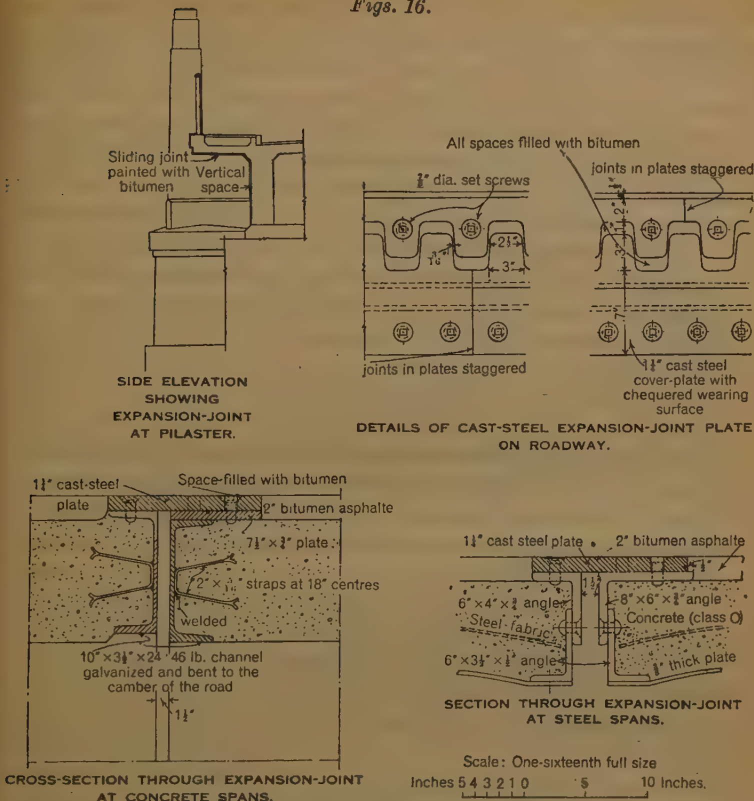
these piers a mild-steel plate sliding on a phosphor-bronze plate fitted to the bedplate on the pier (*Figs. 15*).

Expansion-Joints on Roadway.

The provision of suitable expansion-joints on the roadway of the steel and concrete spans was given full consideration and several alternative designs were prepared before that finally adopted was decided upon. This consists (*Figs. 16*) of two sets of toothed plates formed of cast steel and securely screwed down to galvanized channels or double-angle members fastened to the substructure. The larger toothed casting is carried over the gap between the two sections of the structure and rests on a galvanized plate to which is screwed the smaller toothed casting. The teeth mesh together so that a vehicle passing over is not subject to any jolt. The space between the teeth is filled with bitumen and the castings are provided with a studded surface to prevent slip. Near the curb the meshed teeth terminate and the upstanding portion consists of two parallel plates which extend to the back of the curb. Somewhat similar provision is made on the 50-foot reinforced-concrete spans, the only difference being in the method of fixing to the concrete.

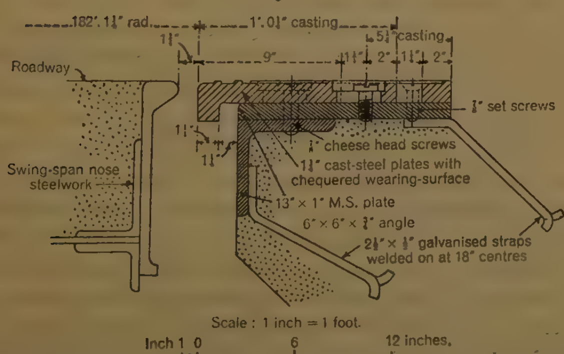
For the curved edging at the swing-span a special arrangement

Figs. 16.



ROADWAY EXPANSION-JOINT DETAILS.

Fig. 17.



ADJUSTABLE ROADWAY PLATE NEXT TO END OF SWING-SPAN.

was adopted (*Fig. 17*, p. 723). Due to the tendency of such structures to increase in length as the years pass by, a process of expansion during hot weather and incomplete retraction afterwards, it was realized that an adjustable section was essential. The end of the roadway on the swing-span is defined by a curved bulb-angle, and parallel to this on each adjacent pier is provided a cast-steel curved plate fixed by screws to a galvanized plate securely tied into the concrete of the roadway at the end piers. By means of slotted holes in the castings the position of the end plate relative to the swing-span can be adjusted by 1¼ inch in addition to the normal gap already provided of 1¼ inch. All these expansion-plates have been found to be very effective in operation.

Swing-Span.

The requirements of the Act necessitated the provision of two navigation openings each 150 feet clear, with a headroom of 40 feet above O.D., or 30 feet above high water. An examination of the relative merits of a rolling lift bridge as compared with a swing-span had been carried out in the early stages of the scheme. It was found, however, that a rolling lift bridge would be a much more costly structure than a swing-bridge, with no particular advantages at the site in question to justify its adoption. The design chosen was, therefore, a symmetrical swing-span to give two clear openings of 150 feet each. The extra opening provided was of great value to the shipping interests.

The superstructure consists of two open girders of the Warren type, each with eight panels of 40 feet 6 inches with a vertical suspender. The centre panel, 34 feet wide, is cross-braced. The overall length of the swing-span over the coamings is 364 feet with a height of 28 feet at the ends and 48½ feet in the centre. The shape of the top boom, a series of straights in the general form of a parabolic curve, was adopted to improve the appearance of the structure as compared with the more usual straight-line top boom. The Warren type of bracing was chosen in place of the more customary N type for similar reasons (*Fig. 18*, facing p. 693).

The width between the main booms is 34 feet to accommodate a 30-foot roadway and to give a 1-foot clearance from curb to edge of the booms, which are 1 foot 10 inches wide. Substantial portal- and wind-bracing of lattice- and built-up-angle construction is provided at main and intermediate panel-points with K-type sway-bracing on the plane of the top booms. A minimum headroom of 18 feet is provided through the structure. The 5-foot-wide footpaths are cantilevered from the outside of the booms in a somewhat similar

manner to that in the remainder of the steel spans. A point to be noted is the provision of a 12-inch by 4-inch channel at a height of 3 feet 6 inches from the roadway alongside the inner edge of each girder to act as a barrier against damage being caused to the main members of the structure by a vehicle mounting the curb.

The floor of the bridge is formed on cross-girders 5 feet deep at 20-foot 3-inch centres, consisting of a $\frac{3}{8}$-inch webplate and 8-inch by 4-inch angles with 10-inch and 17-inch flanges. Longitudinal stringer-joists 20 inches by $6\frac{1}{2}$ inches at 5-foot centres complete the framework of the floor on which the usual $\frac{3}{8}$-inch buckle-plate is provided. Above this is the concrete filling reinforced with steel fabric and finished with 2 inches of bituminous asphalt. The footpaths are of reinforced concrete $5\frac{1}{2}$ inches thick, and in addition to the outer ornamental parapet a separate post-and-tube handrail is provided at the inner edge of the footpath.

In designing the main girders the following variations of loading were taken into account, a combination of the most severe conditions being adopted from (1) and (2), or separately from (1), (3), and (4), as well as from shear conditions in the case of the diagonals :—

(1) Cantilever : dead load only.
(2) Simply-supported beam : live load only.
(3) Continuous beam : live plus dead load.
(4) Continuous beam : dead load plus live load on one span only.

The maximum stresses experienced and the sections adopted for the various members of the structure are given in Appendix II. It will be seen that many of the members are made up with bulb angles. The appearance of the structure is considerably enhanced by the adoption of this section. Special care was taken in the design to permit easy access for future painting, and also to avoid pockets where rainwater could accumulate.

The weight of the superstructure is transmitted by means of two longitudinal and two transverse girders on to a drum-girder with a diameter of 36 feet $9\frac{5}{8}$ inches. The longitudinal girders consist of an extension of the lower boom of the main structure on the central 34-foot panel so as to give a double-webbed girder 8 feet 4 inches deep (Figs. 19, Plate 2). The transverse members each consist of a girder 11 feet deep as shown in Figs. 20, Plate 2. The eight points of support where these girders rest on the drum-girder consist of steel pads $2\frac{7}{8}$ inches thick and 4 feet 9 inches and 2 feet 9 inches long, the fixing being by means of a total of ninety-four fitted bolts $1\frac{5}{16}$-inch diameter. Access-manholes for painting, etc., are provided in the double-webbed longitudinal girder, whilst the

entrance door to the machinery-room, situated between the roadway and the drum-girder, is through the centre of the transverse girder.

The transference of the weight, over 1,500 tons, from eight concentrated points of load to an uniform distribution over the rollers is effected by means of a substantial and very stiff double-webbed drum-girder (Fig. 21, Plate 2). It has a depth of 5 feet and a diameter of 36 feet $9\frac{5}{8}$ inches, with two $\frac{3}{4}$-inch webs separated and stiffened by 10-inch channels at close intervals. The flanges are 8-inch by 6-inch by $\frac{3}{4}$-inch angles with a $\frac{3}{4}$-inch top plate and a $\frac{7}{8}$-inch bottom plate. It was delivered to the site in eight sections covering 45 degrees each. A series of eight open radial girders 3 feet 3 inches deep further stiffen the drum-girder and tie it to a cast-steel cover encircling the centre pivot. The top of these girders is firmly braced by steel joists on which $\frac{7}{16}$-inch-thick chequer plate is riveted to form the machinery-floor.

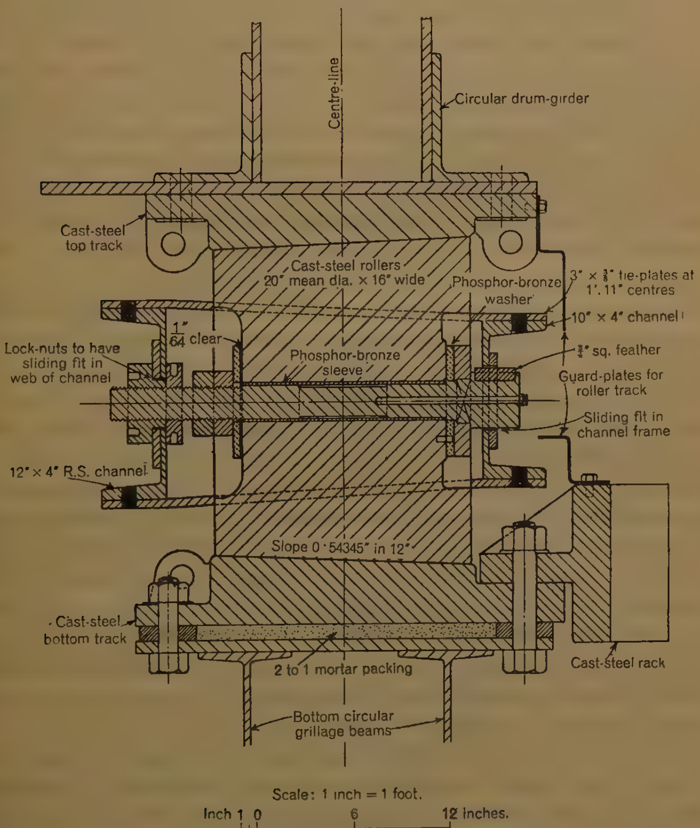
On the lower edge of the drum-girder is fastened a cast-steel tapered roller-path resting on sixty rollers, each of which has a mean diameter of 20 inches and is 16 inches long. The tapering lines of the rollers are arranged to coincide at the centre pivot. These rollers are of solid cast steel, slightly recessed at each end and holed for the spindle. A very reliable set of castings was provided by the makers. The substantial circular cage holding these rollers in position is built up from 10-inch by 4-inch outer, and 12-inch by 4-inch inner, channels, tied together with straps (*Fig. 22*). This cage is held in position by thirty steel radial rods $1\frac{3}{4}$-inch in diameter, fixed at their inner end to a lower cast-steel plate encircling the centre pivot. A turnbuckle is provided to regulate the adjustment of their length. Alternate rollers are held in position by the spindle fixed to the cage only (*Fig. 23*, facing p. 693).

Particular care was taken to ensure that these rollers should rotate with the utmost freedom. Each spindle passing through the centre of the rollers is encased in a phosphor-bronze sleeve which has suitable clearance not only between the spindle and the sleeve but also between the sleeve and the roller, in order to provide a differential effect. The outer end of the spindle is prevented from rotating by a key and feather fitting to the outer channel of the cage. This permits the removal in an outward direction of the whole of the spindle and its sleeve for inspection or renewal of the phosphor-bronze sleeve.

To take up the outward thrust due to the tapered shape of roller, each spindle near the outer edge of the cage has a square neck on which is fitted a mild-steel collar 7 inches in diameter. Against this bears a $\frac{1}{2}$-inch phosphor-bronze plate of similar diameter attached to the roller by means of a pin to ensure rotation. The outward thrust is transmitted in this manner by means of the radial rods

back to the centre pivot. At the inner edge of the roller a similar phosphor-bronze plate is provided with, in this case, a clearance of $\frac{1}{64}$-inch between it and the roller. The cage was designed to be capable of a limited amount of vertical movement to prevent it bringing pressure on to the roller spindles due to any slight irregularity in its level. For this reason the hole in the web of the outer channel

Fig. 22.



Detail of Swing-Span Rollers.

of the cage into which the circular spindle fits is made slightly elliptical, whilst at the inner channel the two nuts on either side of the web are arranged to lock together with a sliding fit only against the web. The sixty rollers were machined to a tolerance of 0·003 inch in diameter and taper. A centre-line was scribed around each roller to facilitate setting at the site. The assumed maximum load

coming on any one roller when rotating was taken at 33 tons to allow for incomplete distribution through the drum-girder.[1] The safe load for rollers in motion ($500dl$, d and l being in inches) is 70 tons. In the fixed position the maximum load assumed to come on one roller is 62 tons. The safe load in this case ($600dl$) is 84 tons. In view of the exposed position of the structure galvanized-iron guards are fitted to prevent snow filling up the spaces between the rollers with serious results. Lubrication of the roller-spindle and phosphor-bronze thrust-plate is provided for by two Tecalemit grease-nipples.

The centre pivot is shown in *Fig. 23* (facing p. 693) and Figs. 24, Plate 2, and consists of a steel casting with a base 7 feet 6 inches in diameter and a height of 7 feet. Two collars with phosphor-bronze liners suitably lubricated rotate on it, the upper collar carrying the eight radial girders which are connected with the drum-girder and support the machinery-floor, whilst the lower collar supports the thirty tie-rods connecting the rollers to the pivot. Through the centre of this pivot are brought the electric cables for the control of the swing-span and for the operation of the machinery.

The lower track supporting the rollers is a steel casting 16 inches wide and machined to suit the taper of the rollers, and is built up of sixteen segments to a diameter of 36 feet $9\frac{5}{8}$ inches. The outer portion of the casting carries the fixed rack by means of which the swing-span is turned. It was appreciated that the easy operation of the bridge depended entirely on the accuracy of the machining and setting out of this roller-path. So far as concerned the machining, it was possible at the sub-contractors' works to have the whole track assembled and machined by a large-radius planing machine. This ensured the accuracy of the machined track within the same limits as those set down for the rollers. A centre-line was also scribed on the track to facilitate the setting out. The base of the castings was rough-machined only, and it was realized that some means for minute vertical adjustment had to be provided for the exact setting of the castings at the site.

A support for the assembled castings was provided by a steel grillage consisting of two 20-inch by $6\frac{1}{2}$-inch joists with a $\frac{3}{4}$-inch top plate holed for grouting. This grillage was set to as close limits for line and level as could be obtained, the greatest variation in true level being of the order of $\frac{1}{32}$ inch. Thereafter it was concreted into position, any space left under the upper plate being grouted up under pressure. This provided a solid and practically level foundation for the setting of the castings.

[1] Number of rollers assumed to carry load under each of eight load-points

$$= \frac{\text{Twice depth of drum-girder}}{\text{Diameter of roller}} = \frac{2 \times 5 \text{ feet}}{1\frac{5}{8} \text{ foot}} = 6 \text{ rollers.}$$

The centre pivot was temporarily placed in position in the exact centre position of the swing-span. This pivot was used to take a radius-rod by means of which the sixteen segments of the roller-path were bolted together and adjusted for concentricity by means of the radius-rod. Tacking bolts were fitted on each segment to hold the castings in position. Approximate levelling was effected at the same time by driving folding wedges at intervals of 6 feet in the 1-inch space between the grillage-plate and the casting. The centre pivot was then removed and replaced by a steel plate with a Watts precise micrometer-level without tripod installed in the exact centre position. Tests were made to determine the variation in level of 1 degree of the micrometer scale, and by this means it was possible to estimate levels to 0·001 inch on a small closely-divided steel engineer's scale set on the castings at the scribed centre-line.

It should be explained that on account of the exposed position of the pier and of the inclement weather when this work was carried out (during February, 1936), a tent was erected to enclose the whole of the track, and it is to this fact that a large measure of the accuracy can be ascribed. The work then consisted of levelling the track and adjusting by screwed bolts and tapered wedges both for level and side taper. The side taper was checked by an engineer's precise block-level set on a steel wedge accurately machined to the exact taper.

It was arranged that wedges 6 inches wide should be fitted in the 1-inch space at 2-foot intervals to fit closely between the grillage-plate and the casting. This was effected by having the bottom folding wedge of a standard thickness, the space to be occupied by the top wedge being measured and the wedge ground to suit the varying requirements. Specially-wide wedges were fitted at the joints in the track-plates, these joints being skewed at 22½ degrees to the radial line. The spacing of the wedges was determined so that the casting would be capable of carrying the load from a roller without any intermediate support from the filling in the 1-inch space. The fitted bolts holding the casting to the grillage had machined washers to fill the 1-inch space. After the whole of the levelling had been completed the centre pivot was replaced and the radius re-checked. Finally, after the track had been accurately levelled and the side slope adjusted, the various bolt-holes were drilled through the casting and grillage-plate and the fitted bolts screwed up. The 1-inch space was grouted up with 2-to-1 mortar made as dry as possible and driven in with steel rammers so as entirely to fill the 1-inch space. Prior to the adoption of this material tests had been carried out on "fusible cement," "rust cement" and other patent preparations. The strength, workability and reliability of the mortar were far in excess of those of all other

mixtures. It may be mentioned that a slab of the mortar used, 4 inches by 4 inches by 1 inch, failed at 12,245 lbs. per square inch after 7 days.

Following this operation, the scribed line on the track and the radius-rod were used to re-locate the centre pivot in its exact position central to the track. The results were as follows :—

Average error in level of lower track : 0·002 inch.
Average error in side slope : 0·003 inch.
Greatest error in concentricity of track relative to centre pivot : $\frac{1}{64}$ inch.

These results, which reflected great credit on the staff at the site, have had their repercussions in the extraordinarily low figures given on p. 748 for the power required to turn the structure.

After the completion of the setting of the centre pivot, the drum-girder and its upper casting were assembled on top of the rollers. Prior to dispatch from their works the contractors had erected the drum-girder upside down with its upper casting tacked in position, and had adjusted the castings by means of the scribed centre-line to an exact circle to facilitate the erection at the site. For assembly at the site the bolts connecting the casting and drum-girder were made hand-tight only in the first place and the castings adjusted by means of shims and a feeler until the casting and the rollers were in close contact. The girder was rotated by man-power to check that close contact was still made between the rollers and the upper track, and finally the fitted bolts were securely bolted in position. Red lead driven in under pressure filled any spaces that existed between the drum-girder flange and the back of the casting.

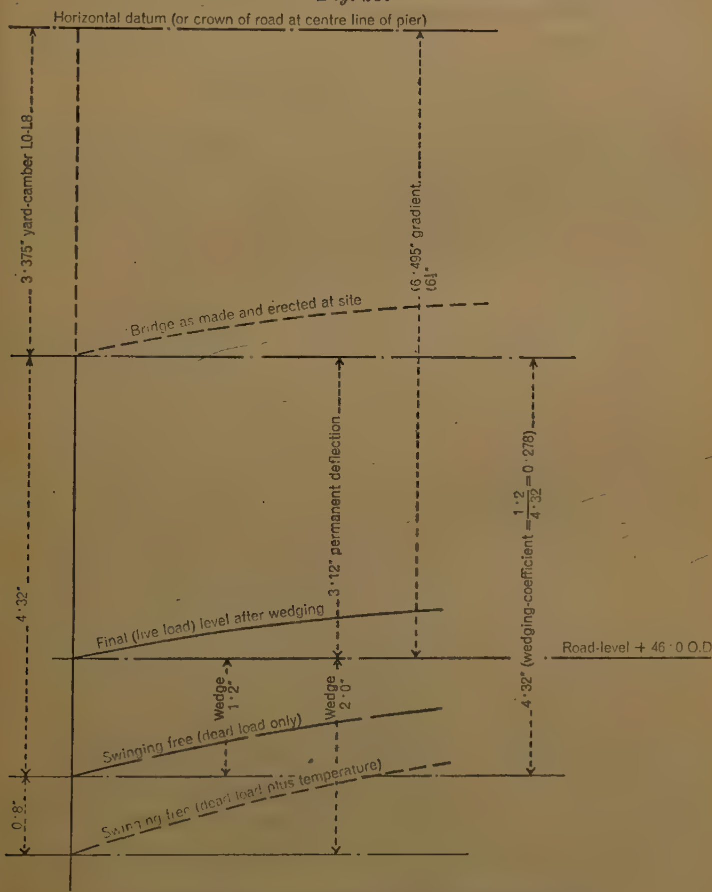
The erection of the centre portion of the swing-span superstructure was easily carried out by means of a 15-ton crane assembled on the temporary staging alongside pier No. 12. To hold the lower booms at their proper level, staging was built up from the deck of the permanent jetty. Certain of the surplus 14-foot 6-inch diameter steel cylinders proved of use in providing a temporary support for this purpose. After a sufficient length had been assembled in this manner a derrick-crane travelling on rails was set on the steel floor at roadway-level and was used for placing the upper chords and end bracing in position.

The entire swing-span had been assembled in position at the makers' works for approval, and this was a large factor in the excellent manner in which the whole structure was found to come together at the site. The roadway of the bridge is constructed with a longitudinal camber of $6\frac{1}{2}$ inches between the centre and the ends. The lower booms were constructed to give $3\frac{3}{8}$ inches camber (Figs. 25,

Plate 2). The theoretical deflexion of the ends below this level with the swing-span free was checked both by graphical methods and by analytical means, and was found to be 5·4 inches.

It was considered that the stiffness due to the gusseted connexions and concrete floor, etc., would reduce the deflexion to about 80 per cent. of the theoretical value, giving a figure of 4·32 inches. In

Fig. 26.



Assumed Deflexions at End of Swing-Span.

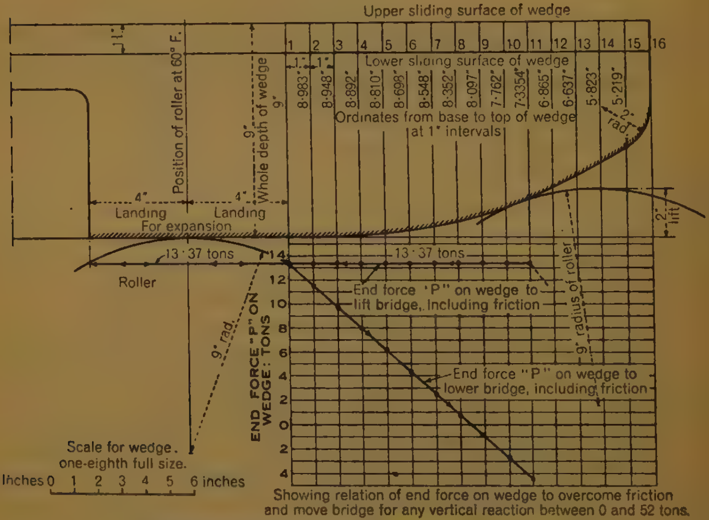
addition a further 1·03 inch theoretical deflexion (0·8 inch actual deflexion) was anticipated from extreme temperature-conditions (*Fig. 26*). As constructed, the deflexion experienced under sunless conditions is 4·3 inches, which shows a remarkable similarity to the assumed deflexion. The greatest temperature-effect on deflexion

has been $\frac{5}{8}$ inch with low-temperature conditions under the bridge due to overnight frost, and with a hot autumn sun heating up the top members. A certain amount of side movement was also experienced (of the order of $\frac{3}{16}$-inch) due to the greater heating of one side of the bridge by the sun. These conditions were obtained with the red-lead shop-coat of paint applied. The result of the application of aluminium paint has been to reduce very considerably the temperature-effects.

Wedges and Locking Bolts.

It was decided to limit the uplift to be given to the ends of the swing-span to an amount just sufficient to avoid "chatter" under the worst loading conditions. This condition arises with one span fully loaded, when the reaction at the far end is reduced to 12 tons

Fig. 27.



Wedge-Profile and Operating Forces.

per girder. An uplift at the ends of 1·2 inch was decided upon to overcome this effect and to give a reasonable reaction of just over 31 tons per girder.

Consideration was given to various types of wedging, such as by screw-jack, hydraulic press, or screw and toggle. The method ultimately adopted was the driving of a wedge, sliding on the underside of the end girders of the swing-span, over a roller set on the supporting piers. The customary shape of wedge for such a purpose

is a straight tapered section, curved off slightly at its junction with the horizontal portion of the wedge. The tapered portion engages first with the roller, and is gradually forced forward to climb up and over the roller until the bridge is supported on the horizontal section of the wedge. The advantages of this type of wedging are many, not the least being that it provides an ideal bearing for allowing the structure to adapt its length to temperature changes.

Since hydraulic power had been decided upon for the operation of the wedges, an investigation was made to determine a more efficient shape of wedge which would produce a practically constant effort for the hydraulic pump to operate against, and a satisfactory shape was obtained. The curved-wedge form adopted (*Fig. 27*) has been found to operate very successfully in practice, giving a gradual lift to the ends of the bridge without jerk and with a steady pump effort over practically the whole of the 27-inch travel of the ram. *Fig. 28* (p. 734) gives a general indication of the wedge with its ram.

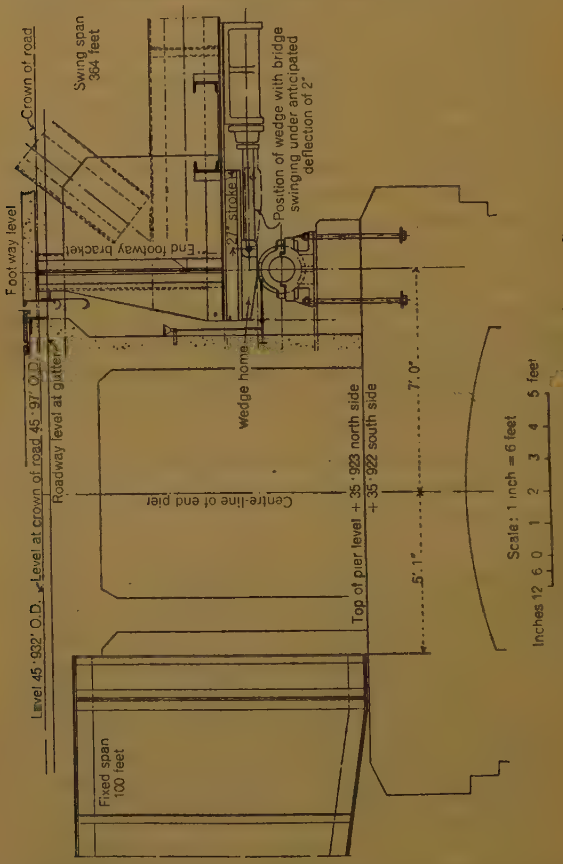
The maximum reaction due to live plus dead load anticipated on each wedge-roller is 184 tons under conditions of 1·2 inch deflexion plus 0·8 inch for additional temperature-deflexion due to a difference in temperature of 20° F. between top and bottom booms. The dead-load reaction on the rollers is 52 tons under these conditions. The forged-steel roller adopted is 18 inches in diameter and 24 inches long. The maximum line-pressure is 7·7 tons per linear inch, compared with a safe stress of 12 tons when at rest (1,500*d*), and is 2·2 tons compared with a safe stress of 8 tons when in motion (1,000*d*).

To ensure that no grooving would take place on either roller or wedge, and especially on the latter where the pressure occurs each time over very narrow limits, the rollers and wedges were heat-treated. The rollers, of special steel, were in addition surface-hardened to a depth of from $\frac{1}{4}$ inch to $\frac{3}{8}$ inch to give a Brinell hardness-number of 248; the wedges were toughened and surface-hardened to give a Brinell hardness-number of 280. They were ground to the special curved profile to within 0·001 inch. The working pressures adopted for the hydraulic equipment are 3,000 lbs. per square inch for the wedge rams and 1,000 lbs. per square inch for the locking bolts, each being capable of being operated under pressures 50 per cent. in excess of this if required.

The locking bolt (one at each end in the centre of the bridge), adopted for centering the structure, is shown in *Figs. 29* (p. 735). This consists of a rectangular steel shaft 6 inches wide and 5 inches deep, with a tapered nose driven by hydraulic power into a cast-steel bevelled socket $6\frac{1}{4}$ inches wide set in each pier on the exact centre-line of the bridge.

The concreting of the roadway and footpaths of the swing-span was carried out in sections of 20 feet, working equally from the centre to the ends so as to maintain equilibrium on the structure, the ends of the span being temporarily supported on jacks in order to give a slight upward reaction. Copper strips were provided in the concrete joints across the footpath and roadway to permit any

Fig. 28.



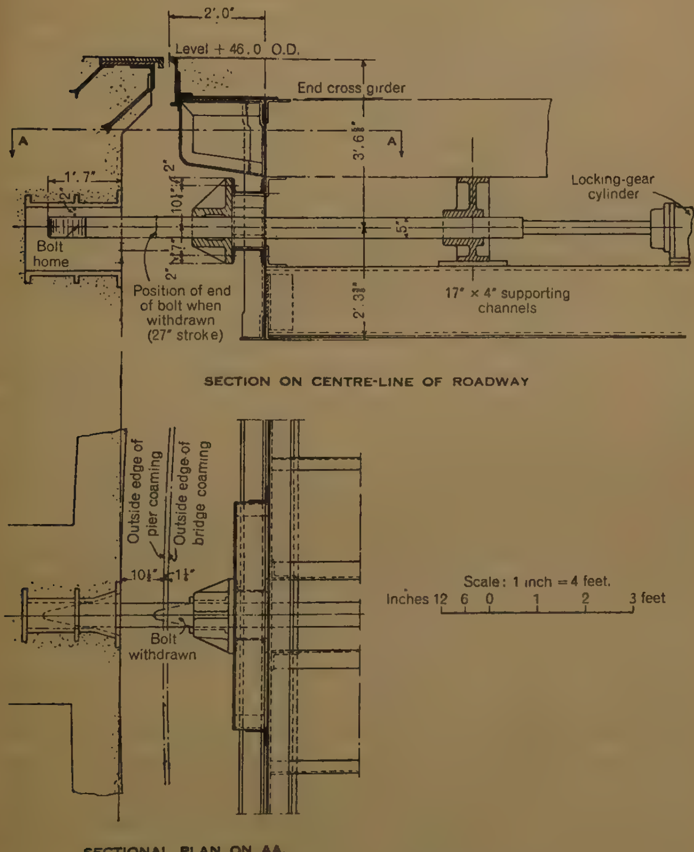
Wedge and Roller at End of Swing-Span.

outward movement under swinging conditions and to avoid cracking of the roadway.

After the actual deflexion of the structure had been ascertained with the roadway and footpaths finally completed, the rollers were accurately set to give the proper uplift to the ends of the structure. All four rollers were set dead level to one another, and the tops of the rollers themselves were also set accurately to level. Th

variation in level of the four wedges at the ends of the bridge when swinging is negligible, thus giving a similar reaction at each corner of the bridge with the wedges driven home. Access to the bolts and wedges for inspection and oiling is obtained through locked

Figs. 29.



LOCKING BOLT AT END OF SWING-SPAN.

manholes at either end of the footpath on the swing span, whilst access to the rollers is obtained through similar manholes on the footpaths at the portals. To reduce the effort required by the operator in lifting these manhole covers they were made of cast aluminium.

Appendix III gives particulars of the various grades of steel and phosphor-bronze used in the rollers and machinery.

TURNING MACHINERY.

The operation of turning the swing-span is effected through a system of rack and pinions driven by electric power. Each of the two separate driving units consists of a 50-HP. 440-volt d.-c. motor running at 600 revolutions per minute and driving, through helical, spur, and bevel gearing, a vertical shaft $9\frac{1}{2}$ inches in diameter, which carries at its lower end a pinion engaging with a rack. The two pinions are set at 180 degrees apart around the rack-circle. The cast-steel rack consists of two hundred and twenty-four machined teeth of $6\frac{3}{4}$ inches pitch and 10 inches across the face, with a mean thickness of tooth of $3\frac{9}{16}$ inch. The rack extends around the whole circumference of the lower track casting, so as to permit the bridge to be rotated in either direction.

The rack-pinion is of specially-toughened forged steel with nine machine-cut teeth 12 inches across the face and of $6\frac{3}{4}$ inches pitch; the pitch-circle diameter is $19\frac{11}{32}$ inches and the outside diameter is $24\frac{19}{32}$ inches with 20-degree involute teeth. Ball thrust-washers take the weight of the shaft with its pinion and bevel gearing. The vertical shaft is supported from the steel structure by very substantial brackets, and provision is made for their exact adjustment by fitting strips on the bearings, no shims being permitted. The horizontal shaft with its mitre passes through the web of the transverse girder, ball thrust-bearings being also provided at this point. The reduction-gearing is supported on a bed-frame of steel joists bolted to the machinery-room floor, bearing being obtained on the radial girders in order to ensure the greatest rigidity.

The reduction-gear, of cast steel with teeth machined from the solid throughout, consists of a series of spur and pinion wheels, the first pinion in the train being connected to the 50-HP. motor through a $3\frac{5}{8}$-inch diameter shaft by means of a flexible coupling. This shaft also carries a 15-inch diameter brake-drum operated through a solenoid. This brake is mainly provided to keep the span in position against wind-forces when in the open position; it is not used under any conditions for slowing down the bridge when decelerating, and is not applied until about 10 seconds after the motors have stopped, the period being adjustable between the limits of 4 seconds and 12 seconds. To reduce noise, a helical gear has been adopted in the first reduction from the motor, this gear in addition running in an oil-tight case. The total reduction is 171 to 1, the rack-pinion making 3·39 revolutions per minute at its maximum speed.

A Ward-Leonard set is provided in the machinery-room. This consists of a 440-volt, 3-phase, 50-cycle a.-c. motor driving a d.-c.

generator giving 180 amperes at 40/440 volts (1-hour rating); the set runs at 970 revolutions per minute. The generator is flexibly coupled to an 8·5-kilowatt exciter generating 40 amperes direct current at 220 volts. This exciter supplies current to all the solenoids and the contactor-coils in the control cabin. The purpose of this equipment is to supply direct current to the main turning motors, which consist of two 50-HP. machines (1-hour rating) operating at 600 revolutions per minute. The speed of turning the bridge is controlled by the voltage-supply fed to these machines, which are in parallel. To ensure that equal torque is exerted by the machines a regulator is installed in the control-cabin for their exact adjustment. The actual turning of the swing-span is carried out in 2 minutes by this equipment, as follows:—acceleration for 10 seconds, travelling at uniform speed for 100 seconds, and deceleration for 10 seconds. The total time required for the complete sequence is 5 minutes approximately. The period that the bridge is closed to road traffic to allow a vessel to pass through is about 13 minutes (minimum).

The electrical supply for the bridge is obtained from the local power-line of the Fife Electric Company, the cable being brought along under the east footpath of the bridge down through pier No. 10 to the river-bed and thence along a trench cut in the bed of the river and up pier No. 11 to a junction-box in the concrete roof of this pier.

To guard against the not infrequent breakdowns of the overhead grid supply, a stand-by diesel engine is also installed in the machinery-room. The principal criterion for this engine was that vibration should be a minimum in view of its being supported on the steel floor of the machinery-house. The type of machine ultimately adopted was a four-cylinder horizontal totally-enclosed forced-lubrication four-stroke engine. This engine is capable of developing a working load of 120 B.HP. for 12 hours at 330 revolutions per minute, and is designed to give an overload of 50 per cent. for 10 minutes.

To reduce further the risk of vibration, the engine is supported on a steel frame bolted to the radial girders which are substantially braced together, whilst beneath the frame a ½-inch layer of felt has been interposed with teak strips between the frame and the engine bed-plate. The air-intake and the exhaust are both fitted with Burgess silencers.

As a result of these precautions, it is practically impossible to detect that the engine is in operation when standing on the roadway of the swing-span, the absence of vibration or sound being extremely marked. Oil and circulating-water tanks are suspended from the underside of the roadway-stringers, their contents being indicated

in the machinery room by Büdenberg gauges. Two compressed-air vessels are installed for the starting of the engine, these being filled by means of a small air-compressor driven by a 3-HP. motor. Arrangements have been made to enable the engine to be started and stopped, as well as for the air-vessels to be charged, by remote control from the cabin 40 feet overhead. Dials in the cabin show the engine-speed, air-pressure, etc. The water in the cooling tanks is circulated by a centrifugal pump driven from the diesel engine.

The diesel engine drives, through Tex ropes, a direct-current generator and an exciter at 1,000 revolutions per minute. The generator gives 90 amperes at 440 volts, and the exciter is interchangeable with that already described for the Ward-Leonard set. By means of a switch in the cabin it is possible to drive the two turning motors in series at half speed from this standby set in order to open the bridge in 4 minutes. For the emergency lighting of the bridge the diesel engine also drives by Tex rope a 35-kilowatt alternator, giving 440 volts at 1,000 revolutions per minute, continuously rated. This will permit the diesel set to be operated at night once or twice a week for lighting the bridge, and will thus ensure that the engine is kept in good condition.

For the hydraulic equipment a 25-HP. motor drives a six-throw high-pressure Tangye pump through helical gearing. The high-pressure rams for the wedges are 1½ inch in diameter and of 4-inch stroke, operating at from 3,000 to 4,500 lbs. per square inch, whilst the rams for the locking bolts are 2½ inches in diameter and of 4-inch stroke, with a working pressure of 1,000 lbs. per square inch. Safety-valves are provided on the delivery pipes on each of the rams, and are at present set at 3,000 lbs. per square inch. The speed of the pump is eighty-four strokes per minute, and the fluid used is oil in order to obviate the risk of the equipment being frozen up during the winter. The actual times and working pressures required for the various hydraulic operations are as follows :—

Driving locking bolts home (27-inch travel) and 500 lbs. per square inch.	35 seconds
Withdrawing locking bolts and 750 lbs. per square inch.	20 ,,
Driving wedges home (27-inch travel) and 1,000 lbs. per square inch.	70 ,,
Withdrawing wedges and 1,500 lbs. per square inch.	40 ,,

The difference in time and pressure between driving and withdrawing is largely due to the smaller cylinder-area being fed with

pressure oil in the withdrawing condition, owing to the presence of the ram.

Since the pump always delivers oil from the pump-ram in the one direction, a valve-operating mechanism had to be provided to obtain the reverse movement of the wedges and bolts. The valve-gear is of stainless steel, the valves being of the mushroom type with long stems to ensure proper guidance on their seats. A spring return is provided. The valves are raised or lowered by a camshaft rotating through a small angle. The stroke of each valve is controlled by tappet switches. The rotation of the camshaft in one direction opens a series of valves for the forward movement of the wedges and bolts, whilst its movement in the reverse direction causes a reversal of flow of the oil through the valve labyrinth. The movement of the camshaft is governed by heavy solenoids. The avoidance of an air-lock in the system is effected by a spring-loaded relief-valve in the pipe delivering oil to the tank.

To safeguard against possible leakage occurring in one of the main pump-rams which would throw the lifting or lowering of the ends of the bridge out of level, a hand pump is installed in the machinery room in addition to the electrical controls for this purpose described later. The pump contains a high- and low-pressure ram operated by a handle, and by this means and a system of valves it is possible to augment the pressure required to any defective ram so as to enable the wedge or locking-bolt affected to operate in unison with the others. This hand pump was also found to be of great assistance during the initial testing of the hydraulic system.

It was considered of the utmost importance that each corner of the swing-span should be lifted or lowered equally to prevent any side-racking of the structure. After consideration of several alternatives the greatest variation in relative horizontal distance between the moving wedges was fixed at 2 inches. The actual height which this represents will vary depending on its position in the travel of 27 inches, as the lift over 2 inches is different at practically any part of the stroke other than on the horizontal portion of the wedge.

It was necessary, therefore, for the operator in the cabin to have visual indication that a wedge was lagging, and also that the pump should stop after an error of 2 inches in the travel of the defective wedge had occurred. The method adopted to obtain this was to attach a potentiometer regulator to each of the wedges, consisting of a nut fastened to the wedge by means of a bracket and engaging with a long coarse-threaded screw rigidly fixed to the structure. The movement of the wedge causes rotation of the screw, one end of which is coupled to an arm passing over a series of resistance-studs, so that the resistance is varied as the arm is moved across each stud. The

regulators are connected together to form the equivalent of a Wheatstone bridge. By this means it is possible to note from a moving needle on the four voltmeters on the control desk the relative movement of each of the four wedges during their travel.

In the event of one wedge lagging behind by 2 inches, then by means of the variation in the potential thus formed, a relay operates on the control-panel and the three other wedges stop, leaving the pump feeding to the defective ram only. If the defective wedge makes up the 2 inches lag within 30 seconds, the other three wedges resume operation, owing to the equalized potential. It is thus possible for the wedging operation to be completed in a series of stops and starts of this nature which indicates to the operator that rectification of the equipment is necessary in the immediate future.

Should the lagging wedge not make up the 2-inch distance in 30 seconds, the pump shuts down and a red indicator on the control desk signifies that the operation in interrupted. The action then to be taken is for the assistant operator to work the hand-pump in the machinery-room so as to assist the defective ram. Duplicate position-indicators for the wedges are provided in the machinery-room to enable the assistant operator to follow the progress of the wedges in their travel. A push-button in the machinery room, when pressed by the assistant operator after the lagging wedge has made up its lag, restarts the sequence. Similar equipment is provided for the locking-bolts, as whilst it is not so important if one bolt lags behind, yet not until they are both fully home will the interlock permit the next step in the operation of the bridge to take place.

The installation of all the equipment already described into the machinery-room 34 feet by 34 feet did not permit too much free space; the whole of the equipment has, however, been conveniently sited, and room was still left for the collector ring. This apparatus is bolted to the top of the cast-steel pivot. It consists of a central steel tube of $6\frac{3}{4}$ inches outside diameter and 7 feet 6 inches long, divided into four flanged sections bolted to each other. The tube has a heavy layer of mica insulation pressed on to it and on this have been shrunk forty-nine brass rings. The four lowest rings are the power rings to supply the main current of 250 amperes, the other rings passing from 2 to 30 amperes depending on their duty. At the top is a horizontal insulated disc which carries eleven rings. These rings each take 2 amperes and are for control- and telephone-circuits. Brushes supported on a stiff channel-structure fixed to the moving span engage on each ring, by means of which current is taken to the various portions of the span. Double brushes are provided to ensure complete contact, the actual brush being held firmly against

the brass ring by springs. The whole of this collector-ring is carried on a ball-race to ensure that no relative movement between the column and its brushes can occur.

A substantial steel housing 8 feet 4 inches in diameter surrounds the whole of this collector-gear. This is to protect the apparatus from dust and damp and also to ensure the safety of the operators. For this purpose a locked entrance-door is provided, with tell-tale indicators which show whether the ring is alive or is dead, other than the pilot-wire to the shore-substation circuit-breaker. Tubular heaters are provided inside the housing to prevent condensation taking place and causing a short circuit.

The roof of the machinery-room is finished off with asbestos sheeting to ensure insulation and to protect the equipment from damp. A large manhole 6 feet 6 inches by 3 feet is provided in the centre of the roadway for the handling of machinery-parts in and out of the machinery-room. A circular runway with a 4-ton hoist-block is provided on the roof the machinery-room for a similar purpose. Ample windows are provided at the four corners and along the roof for the lighting of the machinery-room. Communication between the cabin and the machinery-room is provided by means of loud-speaker equipment.

The electric cables used in the work were paper-insulated wherever possible. The submarine cable, laid in a trench in the bed of the river, is paper-insulated, lead-covered and armoured. The trench also contains a 3-inch flexible jointed steel water-main supplying water to the diesel-engine tanks and to the 130-gallon tank in the roof of the control-cabin for lavatory and other purposes. The General Post Office telephone-cable to the cabin is also laid in this trench. The cables, water-pipe, soil-pipe, etc., are all taken to and from the cabin in the web of one of the main diagonal members. The whole of the wiring in the control cabin is lead-covered V.I.R. cable.

Control Cabin and Operation of Swing-Span.

This cabin, which measures 27 feet 6 inches by 22 feet 6 inches, is situated 30 feet above the roadway in the centre of the span, access being obtained by a steel ladder fitted to one of the diagonal members. The cabin is a steel-framed and plated structure with a flat steel roof covered with asphalt. The maintenance of contactor-panel equipment is largely bound up with the proper housing and heating of the gear, and the efficiency of the bridge-operators is also greatly influenced by attention to their comfort, especially as a 24-hours' service is necessary. For these reasons the installation of an up-to-date cabin with adequate provision for the comfort of the men

was considered as an essential and economical factor in the design. As constructed it provides a mess-room with hot and cold water, electric cooker, etc., as well as separate lavatory accommodation. The steel walls and roof of the cabin have been well insulated and ample window provision has been made on all sides, special care being taken to give an uninterrupted view to the operator at his desk. The control panels are heated by three 2-kilowatt heaters, these being controlled by thermostat in order to maintain a regular temperature. The walls are panelled in oak except behind the contactor-panels where asbestos sheeting has been installed. Asbestos sheeting is also adopted for the ceiling, the floor throughout being of teak.

The governing factors for the operation of the swing-span were :—

(i) The elimination of risk of damage to the structure due to an error from the human element.

(ii) The duplication of essential equipment to minimize possibility of a complete breakdown.

Some of the measures taken with these points in view have already been described. The electrical equipment in the cabin consists of a large control desk at which, by means of moving pointers and coloured lights, the progress of all the essential sequences can be followed by the operator. Three contactor-panels carrying essential control gear are also conveniently located to this desk.

In order to eliminate risk of damage due to human error it was decided to make the entire operation of the structure automatic, and the simplest method of describing the equipment will be to detail the various sequences that occur in opening the bridge to river traffic.

First of all the shore circuit-breaker is closed to obtain the power-supply, and a "re-set" button is pressed, making the equipment ready for immediate use. The operation of a hand-wheel in the centre of the desk is divided into quadrants, the movement of the wheel either clockwise or anti-clockwise in any quadrant determining the direction in which the bridge will swing. Each quadrant has eight intermediate stops or studs, and the movement of the pointer on the wheel causes the following operations to take place :—

Quadrant-stop. The bridge is open to road traffic. Assuming that the bridge is to be opened for a vessel and that the direction of movement of the bridge is to be clockwise, the pointer is moved in a clockwise direction as follows :

Stud 1. This puts the Ward-Leonard set into operation, and closes a relay determining the direction in which the span is to be turned.

Stud 2. This starts up the motor of the hydraulic pump on light load.

Stud 3. The traffic lights at either end of the structure change from green through amber to red. Loud-toned bells ring at each portal. The pedestrian-warning lamps at the footpaths through the portal show red.

At this stage the next operation awaits the assistant bridge-operator, who has descended to the roadway, to ensure that the bridge is clear of vehicles and pedestrians. He is assisted in clearing the structure by loudspeakers operated from the cabin. He presses one of three locked "all clear" switches on the roadway which causes a buzzer to sound in the cabin and permits the next step in the sequence to be set in motion.

Stud 4. The safety gates in the portal commence to lower and the twelve red lights on the gates are lit. After the gates have been lowered to their final position the bells cease. The lower of two horizontal semaphore-arms at each end of the jetty drops to 45 degrees and a red light changes to amber to signify to the approaching vessel that the swinging of the bridge is about to commence. In addition a siren on the cabin roof sounds for 6 seconds.

Stud 5. The four wedges are withdrawn over the rollers, leaving the ends of the bridge swinging free.

Stud 6. The two locking-bolts are withdrawn from the sockets in the piers and the hydraulic pump-motor ceases.

Stud 7. The bridge commences to swing, accelerating for 10 seconds, then moving at a uniform speed for 100 seconds, and finally decelerating for 10 seconds. This sequence is controlled by limit-switches set within the floor of the collector-ring housing. Ten seconds after the bridge has stopped the brakes are applied.

Stud 8. The upper horizontal semaphore-arm at either end of the jetty is lowered and the second amber light shows, and the siren sounds for 2 seconds. This indicates "all clear" to the vessel.

Quadrant-stop. The handle is set in this position for the termination of the opening sequence, which has taken about 5 minutes.

The progress of the sequence of operations is shown by the lighting of the various indicating lamps on the desk. The reversal of the process to close the bridge is mainly similar, and need not be detailed.

By means of a motor-driven drum controller inside the control-

desk all these operations are mechanically controlled except for the necessity to wait for the "all clear" signal from the road. The moving of the controller-handle from the quadrant-stop to stud 8 would ensure the whole of the foregoing operations being carried out in proper sequence and time without interference by the operator. A switch is provided on the desk to short-circuit the "all clear" switch on the roadway in the event of this being found necessary. It is impossible to turn the pointers on the hand-wheel further than the end of the 90-degree quadrant until the operations of turning the bridge to this point are completed. To stop any operation of the sequence it is only necessary to move the handle back to the previous stud.

A position-indicator 18 inches in diameter is provided on the desk, one complete turn of the pointer through 360 degrees being given for a 90-degree turn of the bridge. By this means it is possible to follow clearly the travel of the bridge at any position on the quadrant to the nearest foot. For the accurate centring of the bridge in the road-position preparatory to driving the locking-bolts two systems, in addition to the above indicator, are provided in the form of a photo-electric indicator and visual indicators.

The photo-electric system, which, it is believed, is entirely novel and has never before been used for such a purpose in Great Britain, consists of a box 5 feet long fastened to the end cross-girder at one end only of the swing-span. It contains three photo-electric cells, the end ones near the base and the centre one near the top of the box. Funnels with white interiors guide the light through a mask which occupies the front of the box. In front of this again is a hinged lid with glass windows.

When a beam of light impinges on the photo-electric cell, the cell instantaneously generates a small current, which is amplified by a suitable electric valve and utilized to operate a relay and contactor. On each end-pier, boxes are built into the concrete which contain three groups of three 60-watt lamps at the bottom and one lamp near the top at the centre. Steel masks concentrate the light and permit adjustment to be made. On the control-desk three indicator-lamps coloured red, amber and green, respectively, have been installed. As the bridge swings round towards the closing position the side indicator-lamp lights first, then the centre lamp lights and then the other side lamp lights if the bridge is still swinging. In addition, a bell rings for the further guidance of the operator as each lamp lights. The spacing of the lamps on the piers has been so arranged that if a side and centre indicator-lamp or the centre lamp only is lit, the bridge is sufficiently centred to permit the bolts to be driven. If a side lamp only is showing,

however, the bridge is not properly centred and requires to be inched into position until the centre light is also showing. It may be mentioned that the photo-electric cell, which is of the caesium type, is not subject to any deterioration and has an indefinite length of life. The ordinary amplifying valve has a useful life of several thousand working hours, whilst the lamps, which are operated below their normal voltage, have also a very extended life.

The provision of a final stop-limit switch at the far end of the swing-span was given a considerable amount of attention, but the combination of vertical deflexion and the possibility of considerable variation in the length of the structure due to temperature changes made the accurate working of such a mechanical switch extremely doubtful. In its place, therefore, the photo-electric system was extended to cover this stop-switch, and, as already stated, so long as the centre lamp or two lamps are lit, the relay which shuts down the motors, applies the brakes, and opens the pump-valves for driving the bolts is actuated.

To cover the unlikely case of a breakdown of the photo-cell equipment, a push-button is provided on the control-desk which overrides the above relay. In this case the bolts can be driven and it is necessary for the operator to ensure that the bridge is centred either by the large indicating dial or by visual means.

The visual indicator is constructed as follows. A vertical piano-wire is fastened on the centre-line of the bridge and close to the cabin window, thus forming the back-sight. The fore-sight consists of a circular frame mounted on the top lateral bracing, a space having been cut out of the central gusset-plate for this purpose. The top half of the frame has a sharp pointed rifle-sight for daylight operation, while for night operation a red and green glass separated by a vertical space $\frac{1}{8}$ inch wide is provided. The target consists of a circular opal-glass disk with two vertical gunmetal flats $\frac{5}{16}$-inch apart, this narrow strip only being illuminated from behind by three standard lamps and being fixed to the back of the portal in a circular recess. As the bridge approaches the dead centre, the illuminated strip will show as a green or red light through the fore-sight, depending on which side it is swinging. When exactly centred the white target-light shines brightly through the $\frac{1}{8}$-inch gap of the fore-sight and also shows up the piano-wire in line.

Other operating equipment provided on the control desk is as follows:—

Inching buttons.—These two buttons when depressed lift off the brakes and start up the motors to move the bridge either to the right or to the left. They are required for the accurate centring of the bridge when necessary.

Pilot motors.—A switch is provided which permits one of two

pilot motors which operates the controller drum to be selected. In view of the importance of this drum a duplicate motor was installed.

Shore circuit-breaker.—A switch is provided for the remote turning on of the main power-supply from the substation installed on shore at the north end of the bridge. This is necessary before the operation of turning the bridge is commenced.

Emergency stop.—This stop brings to an immediate standstill whatever operation is being carried out by the controller.

Brake-release button.—The brakes are kept released as long as this button is pressed. It is used when the bridge is to be pulled into the exact centre-line by the locking-bolt.

Bolt-shooting button.—This button is used during high winds after the span has been accurately centred, to prevent it being blown out of line during the 10-second interval before the brakes are normally applied. When pressed it applies the brakes immediately.

To safeguard against the sudden application of the brakes in the event of a complete failure of the power-supply when the bridge is turning, a link mechanism is provided at the brake-drums, kept free by a special solenoid energized by the main alternating-current supply. If the power fails, the link provides a mechanical catch to prevent the brakes being applied.

Portals and Safety-Gates.

At each end of the swing-span on piers Nos. 10 and 12 an architectural feature is introduced by the provision of large concrete portals or archways which span the full width of roadway and footpaths. The soffit of the archway is 18 feet 9 inches above roadway-level, the clear width being 32 feet. Whilst these portals, which bear carved escutcheons of Portland stone illustrating the arms of the three contributing counties of Fife, Stirling and Clackmannan and have heavy fluting courses, strike a definite note in the approach to the swing-span, their purpose is not wholly ornamental. Within each of the portals is housed a safety-gate which is lowered down like a portcullis to shut off both roadway and footpath before the bridge is swung. The gates weigh 3¾ tons each, are constructed to harmonize with the handrailing, and are suspended by means of a counterbalance and Renold chain over sprocket-wheels in the upper structure. The gates are operated by a 3-HP. motor taking 440 volts direct current and driving the sprocket-wheels through worm gearing. Safety-cams are provided on both gate and counterbalance to prevent their collapse on to the roadway in the event of a failure of the main chains. The gates are designed to be lowered in 15 seconds at a speed of 1·2 foot per second until a few feet from the roadway,

when they gradually creep into position. In addition twelve red lights are provided on each gate, which light up as it commences to lower. Emergency hand-gear for operating the gates is also installed within each portal. A microphone is installed in the cabin by means of which the operator warns the public, through loudspeakers, at each end of the swing-span, to stand clear of the gates. The loudspeakers are also of use in issuing instructions to shipping.

To ensure a more attractive finish to the surface of the concrete in the portals and pilasters and to avoid surface crazing, experiments were carried out on various classes of fabric as well as on a suitable glue for attaching it to the shuttering. As a result the material finally adopted was a wide-mesh hessian canvas similar to that used for backing rugs. The glue found to be most resistant to the action of wet concrete was a cold-water casein glue. This method of surface-treatment required considerable care in the assembly of the shuttering and canvas, as well as in the deposition of the concrete, but the results obtained are held to justify its adoption as an effort to break away from the more conventional form of surface-treatment.

The hand-railing adopted was made as attractive as possible and is 4 feet 6 inches high to ensure adequate protection to pedestrians during high winds; it consists mainly of mild-steel flats $1\frac{1}{2}$ inch by $\frac{1}{2}$ inch. The hand-rail itself is a $2\frac{3}{4}$-inch diameter steel tube flattened to a D shape and bolted to the railing. The ends of each 10-foot section of the hand-rail are sealed by welded strips to eliminate internal corrosion. There is a cast-iron square-section lamp-standard on top of each pier about 18 feet above roadway-level, which is a definite advantage for the navigation of small vessels under the bridge at night. The lamps have 20-inch diameter Morocco-glass globes. Fourteen lamp-standards with circular opal globes lit between sunset and sunrise are provided to define the main jetty and the two fender-jetties. On the lower boom of the swing-span a red warning light is installed at each end, whilst a white light is provided to define the exact centre of the navigation opening. Pilot lamps on the control-desk are provided for the essential navigation and traffic lights, which indicate any failure. The roadway lamps in the middle of the swing-span are provided with a cut-off of about 65 degrees to ensure that no dazzle is caused to small vessels passing under the swing-span. The minimum illumination on the roadway is 0·05 foot-candle, this being the minimum recommended for street lighting. The lamps are either 300- or 500-watt, each lantern being fitted with a non-axial asymmetric prismatic-glass refractor to direct the light on to the roadway.

Power-Consumption in Moving Bridge.

After the bridge had been opened to traffic a check was made on the amount of power consumed in the operation of moving the swing-span. For one complete operation, comprising two turns of the structure through 90 degrees, the total power consumed for all purposes, including main motors, hydraulic-pump motors, gate-operating motors, etc., is 2·1 units, which at $\frac{3}{4}$*d.* per unit represents an expenditure of about $1\frac{1}{2}$d. to open and close the bridge for each ship. For the movement of the swing-span itself the maximum power-demand momentarily reached during the accelerating period is 50 HP., the average power required being about 12 HP.

Based on the measurements of power-consumption, the following particulars of friction, etc., have been obtained. These compare very favourably with those for any bridges of similar type constructed in Great Britain or America.

Effort to turn bridge per 1,000 lbs. of dead weight, 3·45 lbs.

Total friction in turning bridge as proportion of superimposed weight, 0·0037.

Friction in rollers, 0·00178.

Total power to open and close swing-span, 2·1 units.

Power required per ton of load, 0·00131 unit.

Power required per ton per degree of angular movement, 0·0000073 unit.

Paint.

Aluminium paint is employed for the exposed steelwork. This paint has been extensively used for bridges in the United States and in Canada, but its use in Great Britain has been confined to types of outdoor structures other than bridges. At the commencement of the contract work in 1934 samples of aluminium and other paint were obtained from about a dozen specialist firms, and two dozen plates, painted by the same person under practically identical conditions, were established at the site. After exposure for about 2 years it was found that aluminium paint withstood the atmospheric conditions exceptionally well and confirmed the confidence which had been put in this material.

Accordingly the whole of the exposed steelwork is painted with three undercoats, red lead, graphite, and graphite aluminium, the final coat for the external girders, hand-railing, swing-span, etc., being bright aluminium. Underneath the bridge a grey finishing coat has been adopted, whilst the hidden parts of the structure, such

as those under the machinery-room, are painted with a black bituminous paint.

Connecting Roads.

South approach road.—This road, 2,200 yards long, connects the south end of the bridge with the existing Grangemouth—Stirling road. The new road consists, for a considerable portion of its length, of a widening of the narrow existing road to the passenger ferry. The road now provided is 30 feet wide with one 6-foot footpath and consists of 12 inches of hand-set whinstone bottoming and 3 inches of water-bound macadam. A final covering of 3 inches of bituminous macadam with a $\frac{3}{4}$-inch carpet-coat was laid about a year after the first construction of the road. The road passes over the saltings to connect with the bridge, and was founded on 50 feet of soft mud. Stone banks with pitched face-work were formed to resist outward movement of the embankment. Special care was taken not to disturb the upper layer of coarse grass growing on the saltings owing to the support which it provided for the embankment. Over a period of $2\frac{1}{2}$ years a total settlement of 4 feet occurred on a bank 12 feet high, about two-thirds of this settlement occurring shortly after the deposition of the material. Records of settlement were kept throughout and show that the final consolidation has now been practically reached.

North approach road.—This road, 500 yards long, connects with the Dunfermline—Alloa road. The roadway on the embankment next to the bridge, where the maximum height is 17 feet, has been constructed of 12 inches of hand-set bottoming and 4 inches of bituminous macadam. A reinforced-concrete retaining wall of L-shape section with concrete access-stairs is provided on the one side of this road. Elsewhere the road consists of a reinforced-concrete slab 8 inches thick, laid in lengths of 60 feet with a longitudinal joint down the centre, the transverse joints not being staggered but meeting at a common point. The footpaths are 8 feet wide. Normally the road is 30 feet wide, but for the reverse curves of 520 feet radius connecting it to the bridge the roadway is widened to $32\frac{1}{4}$ feet with a super-elevation of 18 inches, transitions being provided on both plan and elevation.

By-pass road.—This road, 700 yards long, provided a direct route for east to west traffic to by-pass the existing narrow and tortuous streets through Kincardine. This is also a 30-foot reinforced-concrete road constructed generally as described above, with two footpaths finished with tarmacadam. The curved portion of the roadway is similarly widened and super-elevated.

Expenditure.

The total cost of the scheme is £322,500, made up as shown in the following Table :

Description.	Engineers' estimate : £.	Final cost : £.
Bridge foundations . . . . Bridge superstructure, auxiliary machinery, etc. .	240,600	241,000
South approach road . . . . Surfacing of south approach road . . . . . North approach road . . . . By-pass road	50,400	35,500
Land, engineering, inspection, legal, preliminary, and promotion expenses, etc.	51,500	46,000
Total .	£342,500	£322,500

It is seen that this is fully £60,000 less than the original estimate of £385,000 made up in 1930. For comparison the modified estimate, £342,500, given to the Ministry of Transport in 1933, is also noted.

The cost of the bridge per square foot of area, including that occupied by the footpaths, is £2 5*s*. The cost per mile of the approach roads is as follows :—south approach road £15,000, by-pass road £15,000. An additional 15 per cent. should be added to these figures if it is desired to include the engineering, legal, land, and promotion expenses. Typical rates for steel, concrete, etc., are given in Appendix IV (p. 758).

Conclusion.

The bridge was designed and its construction was carried out under the direction of the Joint Committee's Consulting Engineers, Sir Alexander Gibb & Partners.

Sir Alexander Gibb, G.B.E., C.B., F.R.S., President Inst. C.E., and Sir Leopold Savile, K.C.B., M.Inst.C.E., were directly concerned with the scheme throughout. The Author has been closely associated with the project since 1930, both in the preliminary stages and promotion, and in the design, and was Chief Engineer for the Consultants from the commencement of the construction in 1933. Mr. R. G. Edkins, B.A., Assoc. M. Inst. C.E., was Resident Engineer throughout, assisted by Messrs. L. G. Booen, B.Sc., J. B. B. Newton, B.Sc., Assoc. MM. Inst. C.E., J. W. Ward, B.A., Stud. Inst. C.E., and Capt. S. A. Stewart. Mr. F. L. Wake was chief

structural draughtsman on the design, Mr. H. F. Bull, Assoc. M. Inst. C.E., dealing with the reinforced-concrete section.

The Ministry of Transport, both in London and in Edinburgh, gave the utmost assistance in connexion with the design and construction of the scheme. Thanks are also due to Mr. James Miller, the architectural member of the Royal Fine Art Commission for Scotland, for advice in regard to the design of the portals and pilasters.

The Contractors for the bridge-foundations and superstructure were the Cleveland Bridge & Engineering Company, Ltd.; Mr. E. W. Gill, O.B.E., M. Inst. C.E., was Chief Engineer, Mr. H. P. Budgen, D.Sc., M. Inst. C.E., being responsible for the electrical and mechanical equipment. Mr. A. Rhodes was Agent at the site. A list of the firms who acted as sub-contractors on the work, as well as of the contractors for the approach roads, etc., is given in Appendix V (p. 759).

The Author desires to pay tribute to the exceptionally fine quality of workmanship given by the principal contractors for this bridge. They were actuated throughout by the desire to produce a structure of the very highest standard, worthy of its position as neighbour to the famous railway bridge at Queensferry.

Of the Sub-Contractors, mention must be made of the General Electric Company, Ltd., for the skilful manner in which they devised special equipment to meet all the requirements specified for a fool-proof design of control for the swing-span, as well as for the high standard of workmanship given.

The Author's thanks are due to Sir Alexander Gibb & Partners for their permission to present this Paper and for the facilities given for its preparation.

The Paper is accompanied by twenty-nine sheets of drawings and by eleven photographs, from some of which Plates 1 and 2, the Figures in the text, and the half-tone page-plate have been prepared, and by the following five Appendixes.

APPENDIX I.

DETAILS OF FOUNDATION-LEVELS OF PIERS, DATES OF CONSTRUCTION, ETC.

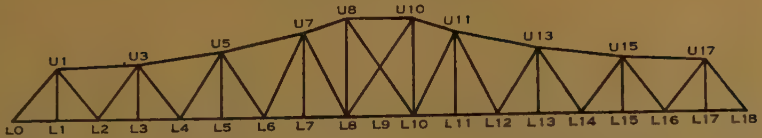
Pier.	Description of foundations and method of excavation.	Level of foundations below O.D.: feet.	Average level of pile-points below O.D.: feet.	Date when commenced.	Date when completed.
		North Side of River.			
North abutment	Piled	—	+ 2 to — 0·5	May, 1934	Mar. 1936
Pier No. 1	,,	—	+ 0·3 to — 0·8	June ,,	May, 1935
,, ,, 2	,,	—	— 9 to — 10	June ,,	May ,,
,, ,, 3	Rock (open excavation)	— 14 to — 17	—	Apr. ,,	May ,,
,, ,, 4	,,	— 13 to — 15·5	—	May ,,	Apr. ,,
,, ,, 5	,,	— 13·5 to — 16	—	July ,,	Mar. ,,
,, ,, 6	Rock (compressed air)	— 24·5 to — 26	—	Jan. 1935	Mar. ,,
,, ,, 7	,,	— 24·5 to — 25	—	Dec. 1934	Feb. ,,
,, ,, 8	,,	— 26·5 to — 29·5	—	Nov. ,,	Jan. ,,
,, ,, 9	,,	— 21·8 to — 22·9	—	Oct. ,,	Dec. 1934
,, ,, 10	,,	— 25·8 to — 27	—	Sept. 1935	June, 1936
Main pier No. 11	,,	— 37·5 to — 38·5	—	June ,,	Dec. 1935

APPENDIX I.—*continued.*

Pier.	Description of foundations and method of excavation.	Level of foundations below O.D. : feet.	Average level of pile-points below O.D. : feet.	Date when commenced.	Date when completed.
		South Side of River.			
Pier No. 12	Piled	—	— 42·5 to — 47	Mar. 1936	June, 1936
,, ,, 13	,,	—	— 52 to — 55	Apr. ,,	June ,,
,, ,, 14	,,	—	— 53·5 to — 55	Sept. 1935	Jan. ,,
,, ,, 15	,,	—	— 57 to — 65	May ,,	Sept. 1935
,, ,, 16	,,	—	— 58 to — 69	May ,,	Oct. ,,
,, ,, 17	,,	—	— 38 to — 63	April ,,	June ,,
,, ,, 18	,,	—	— 38 to — 60	Mar. ,,	June ,,
,, ,, 19	,,	—	— 39 to — 56·5	Jan. ,,	Sept. ,,
,, ,, 20	,,	—	— 42·5 to — 50	Dec. 1934	May ,,
,, ,, 21	,,	—	— 42 to — 55	Nov. ,,	Apr. ,,
,, ,, 22	,,	—	— 42 to — 57	Nov. ,,	Apr. ,,
,, ,, 23	,,	—	— 39 to — 45	Nov. ,,	Mar. ,,
,, ,, 24	,,	—	— 45 to — 46	Nov. ,,	Mar. ,,
,, ,, 25	,,	—	— 45 to — 47	Nov. ,,	Feb. ,,
,, ,, 26	,,	—	— 44 to — 46	Sept. ,,	Jan. ,,
,, ,, 27	,,	—	— 43 to — 45	Sept. ,,	Jan. ,,
,, ,, 28	,,	—	— 43 to — 45	Oct. ,,	Feb. ,,
North end : piled viaduct	,,	—	— 43 to — 50	Oct. ,,	—
South end : piled viaduct	,,	—	— 38 to — 40	July ,,	—

APPENDIX II.

Swing-Span Scantlings.

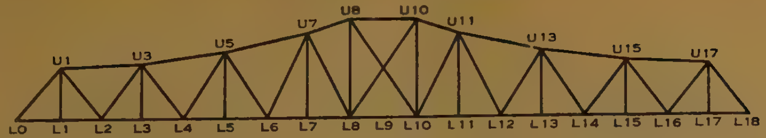


	Member.	Stress : * tons.	Section provided.	
Top boom.	L0U1 L18U17	+ 220		Two plates 22″ × $\frac{7}{16}$″ web. One plate 22″ × $\frac{3}{8}$ top. Two angles 6″ × 4″ × $\frac{3}{8}$″. Two angles 4″ × 4″ × $\frac{3}{8}$″.
	U1U3 U15U17	+ 200		Two plates 27″ × $\frac{7}{16}$″ web. One plate 33″ × $\frac{3}{8}$″ top. Four angles 4″ × 4″ × $\frac{1}{2}$″.
	U3U5 U13U15	+ 189 − 253	ditto	Two plates 27″ × $\frac{7}{16}$″ web. One plate 33″ × $\frac{3}{8}$″ top. Four angles 4″ × 4″ × $\frac{1}{2}$.
	U5U7 U11U13	− 462		Two plates 27″ × $\frac{7}{16}$″ web. Two plates 19″ × $\frac{1}{2}$″ web. One plate 33″ × $\frac{3}{8}$″ top. Four angles 4″ × 4″ × $\frac{1}{2}$″.

Top boom.	U7U8 U10U11	— 646		Two plates 27″ × ½″ web. Two plates 19″ × ½″ web. Two plates 33″ × ½″ top. Four angles 4″ × 4″ × ½″
	U8U10	— 624	ditto	,, ,,
Bottom boom.	L0L2 L16L18	— 140		Two plates 27″ × $\frac{7}{16}$″ web. Four angles 6″ × 4″ × ½″.
	L2L4 L14L16	+ 165 — 193	ditto	,, ,,
	L4L6 L12L14	+ 358	ditto	Two plates 27″ × ¾″ web. Four angles 6″ × 4″ × ½″.
	L6L8 L10L12	+ 544		Two plates 27″ × ¾″ web. Two plates 15″ × ½″. One plate 22″ × ⅜″ top. Four angles 6″ × 4″ × ½″.
	L8L10	+ 624	ditto	Longitudinal distributing-girder.

* + denotes compression; — denotes tension.

APPENDIX II.—*continued.*



	Member.		Stress: * tons.	Section provided.	
Verticals.	U1L1 U5L5 U11L11 U15L15	U3L3 U7L7 U13L13 U17L17	− 75		22″ × 7″ R.S.J.
Verticals.	U8L8	U10L10	+ 190		One plate 22″ × $\frac{3}{8}$″. Four bulb angles 8″ × $3\frac{1}{2}$″.
Diagonals.	U1L2	L16U17	+ 100 − 140	ditto	One plate 22″ × $\frac{3}{8}$″ Four bulb angles 10″ × $3\frac{1}{2}$″.
Diagonals.	L2U3	U15L16	+ 139 − 138	ditto	” ”
Diagonals.	U3L4	L14U15	+ 196	ditto	” ”

Diagonals.	L4U5	U13L14	− 244	ditto	,, ,,
	U5L6	L12U13	+ 286	ditto	One plate 22″ × $\frac{3}{8}$″. Four bulb angles 12″ × $3\frac{1}{2}$″.
	L6U7	U11L12	− 345	ditto	One plate 22″ × $\frac{1}{2}$″. Four bulb angles 12″ × $3\frac{1}{2}$″.
	U7L8	L10U11	+ 402		One plate 21″ × $\frac{5}{8}$″. Two plates $23\frac{1}{2}$″ × $\frac{1}{2}$″. Four bulb angles 12″ × $3\frac{1}{2}$″.
	U8L10	L8U10	− 51 + 128		One plate 22″ × $\frac{3}{8}$″. Four bulb angles 10″ × $3\frac{1}{2}$″.

* + denotes compression; − denotes tension.

APPENDIX III.

Materials used in Rollers, Machinery, etc.

Tapered rollers and roller-tracks: cast steel B.S.S. No. 30 (1907), Grade A. Ultimate tensile strength 35–40 tons per square inch.

Rack: cast steel B.S.S. No. 30, Grade A.

Centre-pivot: cast steel B.S.S. No. 30, Grade B. Ultimate tensile strength 26–35 tons per square inch.

Pinions: forged steel B.S.S. No. 24 (Part 4). Specification No. 9 (1930), Quality D. Ultimate tensile strength 40–45 tons per square inch.

Steel shafting and pins: B.S.S. No. 29. Ultimate tensile strength 32–36 tons per square inch.

Machinery-castings: B.S.S. No. 30 (1907). Gear-wheels of Grade A steel. Other castings of Grade B steel, ultimate tensile strength 26–35 tons per square inch.

Bronze machinery-bearings: copper 80 per cent., tin 10 per cent., lead 10 per cent.

Wedges and rollers: forged steel B.S.S. No. 24 (Part 4). Specification No. 9 (1930), Class D quality. Brinell hardness numbers: rollers 248; wedges 280.

Castings for hydraulic gear: B.S.S. No. 30 (1907), Grade B steel.

Castings for rocker bearings: B.S.S. No. 30, Grade A.

Bronze sliding bearings: Immadium bronze (The Manganese Bronze & Brass Company, Ltd.).

APPENDIX IV.

Typical Rates.

Foundations Contract.

Steel in 14-foot 6-inch diameter cylinders	£27 per ton.
Concrete (class E)	40*s*. per cubic yard.
Concrete (class F) in facing to piers	1*s*. 7*d*. per cubic foot.
Concrete (class D) in beams	1*s*. 6*d*. per cubic foot.
Steel reinforcement	23*s*. per cwt.
Shuttering for beams and slabs	9½*d*. per square foot.
Creosoted timber, 14 inches by 14 inches	5*s*. 6*d*. per cubic foot.

Steelwork Contract.

Steel in girders	£23 5*s*. per ton.
Steel in swing-span	£24 17*s*. 6*d*. per ton.
Cast steel	£60 per ton.

APPENDIX V.

Principal Contractors.

Foundations and steelwork	The Cleveland Bridge & Engineering Co., Ltd.

Sub-Contractors.

Cement	Tunnel Cement Co., London. Cement Marketing Co., London.
Creosoted timber	Denny, Mott & Dickson, Ltd., London.
Steel castings: Tapered rollers, etc.	Kryn & Lahy, Ltd., Letchworth.
Steel castings: Expansion plates, etc.	Head Wrightson & Co., Ltd., Thornaby-on-Tees.
Swing-span turntable	Sir Wm. Arrol & Co., Ltd., Glasgow.
Turning machinery	J. M. Henderson & Co., Ltd., Aberdeen.
Electric motors	Laurence Scott & Electromotors, Ltd., Norwich.
Hydraulic equipment	Tangyes, Ltd., Birmingham.
Rollers and wedges	Walter Somers, Ltd., Halesowen.
Cast-iron gullies, etc.	Mackenzie & Moncur, Ltd., Edinburgh.
Electric control and lighting equipment	The General Electric Co., Ltd., Birmingham.
Cable-laying	Edmiston Brown & Co., Ltd., Glasgow.
Diesel engine	Brush Electrical Engineering Co., Ltd., Loughborough.
Safety-gate machinery	M. B. Wild & Son, Birmingham.
Water-pipes	Stewarts & Lloyds, Ltd., Glasgow. Glenfield & Kennedy, Ltd., Kilmarnock. Stanton Iron Works Co., Ltd., Nottingham.
Portland-stone escutcheons	Joseph Armitage, London.
Granite curbs	The Dalbeattie Granite Co., Ltd., Dalbeattie.
Road-surfacing	John Miller & Co., Inverkeithing.
Lamp-posts	Bromsgrove Guild, Ltd., Bromsgrove.
Control-cabin joinerwork	Scott Morton, Ltd., Edinburgh.
Control-cabin windows	Henry Hope & Sons, Ltd., Birmingham.
Aluminium paint, etc.	J. Dampney & Co., Ltd., Newcastle-on-Tyne.

Approach Roads—Contractors

South approach road	A. A. Stuart & Sons, Ltd., Glasgow.
South approach road surfacing	Geo. Wimpey & Co., Ltd., Edinburgh.
North approach road	George Bald, Dunfermline.
Kincardine by-pass road "Fere Gait"	J. Baxter & Sons, Dunfermline.

Discussion.

The Author. The AUTHOR exhibited a number of lantern-slides illustrating h Paper.

The President. THE PRESIDENT observed that the Paper gave full details of a the work that had been done, and it would be very useful for futu reference. He would not say very much about the bridge, as tim was short and there were other speakers. It was an example efficient and cheap construction. He had taken out the costs p square foot of various road bridges in London; two of the cheape of them had been Kew bridge—a granite bridge—which had co £10 13*s*. per square foot, and Vauxhall bridge, which had cost £7 15 per square foot. The Kincardine bridge had cost £2 5*s*. per squa foot. He had brought up the costs of the Vauxhall and Kew bridg to present-day prices, so as to give a fair comparison, and had omitt costs of land, lawyers' fees, engineering fees, and approaches in cases.

To his mind the principal reason for the low cost of the Kincardi bridge was the whole-hearted co-operation of all those concerne In the first place, Messrs. Mott, Hay and Anderson had reported the project for the bridge, and had shown the advantage of t Kincardine site. Next, Mr. F. C. Cook and the local engineers of t Ministry of Transport had been extraordinarily helpful, and witho them the work could not have been carried out. The Coun and Burgh Councils that had subscribed the money had always be most reasonable, and ready to accept any sensible suggesti The Contractors had taken the contract when times had been ba and the cost of their tender had been rather low, but they h adhered to their prices and had carried through the work in a m satisfactory manner. The success of the bridge had been ve largely due to the excellence of their work and of that of their vari sub-contractors. The work had been done for 6 per cent. below t estimate which the engineers had given before the contract had be let, and 17 per cent. below the figure on the basis of wh the Ministry of Transport had decided to proceed with the proje The country therefore owed a good deal to the contractors and the engineers. He wished to express his own thanks to the Aut in particular, who had been responsible for the work, had d it extraordinarily well, and had given The Institution a first-r Paper upon it.

Sir Leopold Savile.

Sir LEOPOLD SAVILE observed that on the north side of the river there had been suitable rock for foundations from the shore to the centre pier. After that there had been no rock, and the foundations had had to be carried on piles driven down to the ballast. When the work had commenced the only borings that had been taken near the fault were one at about the centre of the central pier, whose position had had to be settled beforehand, and one about 200 feet further south; as the central pier was about 50 feet in diameter there had been some doubt whether it would be entirely on solid rock or possibly on the edge of a cliff. However, when additional borings had been taken it was found that the foundations of the pier would be on good sound rock, so the question, which had caused a good deal of anxiety at one time, had been satisfactorily settled.

For opening the bridge and shutting it again only 2 units of electric power were required; the price was $\frac{3}{4}d.$ per unit, so that the cost of current for opening and shutting the bridge was $1\frac{1}{2}$d. The average number of openings during the year was 600, so that the total cost of electricity for operating the bridge was under £4 per annum—a remarkably low figure.

Dr. Anderson.

Dr. DAVID ANDERSON remarked that before dealing with one or two points in the Paper he wished to digress for a moment by putting in a plea for shorter Papers. The Institution spent about £12,000 per annum on its publications, of which £10,000 per annum might be taken as representing the cost of issuing the Journal. A Paper of average length—about 40 pages—cost about £500 to publish; the present Paper, however, was practically twice the average length, so that it would cost about £1,000.* He did not want The Institution to cut down its printing bill unduly, but he did suggest that Authors of Papers should concentrate on what was novel. There was a good deal which was novel in the present Paper, but there were certain parts which were more or less standard practice, and Authors should cut down such matter as much as they could, bearing in mind that printing was expensive and that it was very difficult to ask an Author, after he had gone through the hard work of writing a Paper, to cut it down. It would be far better to print two concise Papers than a single bulky one.

In the original report on the Kincardine project (and also in the report that his firm had prepared on the Alloa bridge) it had been laid down that the opening span should consist of two openings of

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* The figures given above were only a rough approximation. More accurate figures indicated that a Paper as submitted by an Author cost from £250 to £500 to publish, according to its length; the addition of the oral and written discussion increased that cost to from £350 to £650.—D. A.

Dr. Anderson. 100 feet. That had since been increased to two openings of 150 feet and he would like the Author to state the reason for that increase i span. The river traffic was quite small—about one ship per day and a single opening of 150 feet had sufficed during construction would not that opening have sufficed permanently? On p. 724 th Author stated that a rolling lift bridge had been considered in th early stages of the scheme, but had been found to be more costl than a swing span. When the span had been increased, had fresh study been made of some other type of opening span instea of a swing span? A study of the Bangkok Memorial bridge designed by Messrs. Dorman, Long and Company would appear t show that a bascule opening might have been economical. Th Bangkok bridge had a roadway of practically the same width a that of the Kincardine bridge, but provided a single clear opening o 196 feet. The total moving weight was 1,420 tons, so that for a singl opening of 150 feet it might be expected to be between 700 and 1,00 tons, instead of 1,600 tons, as in the Kincardine bridge. Admittedly he himself had suggested two 100-foot spans for Alloa, but th change from 100 feet to 150 feet clear opening was attended with very considerable increase in cost.

One of the novel features mentioned in the Paper was the drivin of piles under the cylinders. So far as he knew, it was the firs instance in Great Britain of the adoption of that type of constructio and he desired to congratulate the engineers on its use. He though that the choice was justified, particularly by the very low cost o the bridge, to which the President had drawn attention. He woul ask the Author to divide the very creditable average figure of £2 5 per square foot into one figure for the fixed span and another figu for the swing-span.

It appeared that the obstruction caused by the piers was 1 per cent. both at high water and low water, which was a highe figure than usual; it was also stated that the river was a swif flowing one. He would like to ask whether any scour was takin place.

It was stated on p. 737 that the interruption to road traffic when th bridge was opened was 13 minutes. Fortunately that only occurre once a day, but 13 minutes was a long time. It was apparent made up of 2 minutes for opening the span, 2 minutes for shutti it, and 3 minutes for dealing with the various locking gears, leavi about 6 minutes' delay due to river-traffic, which was beyond th control of the engineer. Some time ago he had inquired of vario

[1] F. W. Thompson, "The Mechanical Gear of Bangkok Memorial Brid Siam." Inst. C.E., Selected Engineering Paper No. 149 (1933).

uthorities what average delay took place at other opening bridges, Dr. Anderson.
nd the replies were as follows :—

Tower bridge, $4\frac{1}{2}$ minutes.
Newcastle swing-bridge, 7 minutes.
Boothferry swing-bridge, $12\frac{1}{2}$ minutes.
Queensferry Scherzer bridge, 8 to 10 minutes.
Keadby Scherzer bridge (a railway bridge), 4 to 6 minutes.
Tees (Newport) vertical lift bridge, 8 to 10 minutes.

Ie thought that a fair comment on those figures was that river traffic ould be and ought to be hustled ; more consideration might well be aid to the general benefit of the community by ensuring as brief a olding-up as possible of road traffic in connexion with opening ridges.

Sir CHARLES H. BRESSEY mentioned that at an early stage he had Sir Charles Bressey. een associated with the negotiations which had led up to the contruction of the bridge, although he had subsequently transferred hat responsibility to Mr. Cook. In the course of those negotiations which had been by no means easy) he had been very much impressed y the amazing tact, patience and skill in negotiation of Lord Elgin. ord Elgin had had the difficult task of driving a team consisting of hree Scottish County Councils, two Scottish Burghs, the Ministry of 'ransport, and, in the background, another Government department ontrolling national finance, and he was entitled to the greatest credit or his work. He suspected that Lord Elgin had wished that he could ave carried the negotiations through rather faster, because at one ime the Treasury had offered an 85-per-cent. grant. Then had come he period of depression, and the grant had been reduced to 75 per ent. However, Lord Elgin had overcome all the obstacles.

The unusual length of the Paper was probably due to the Author's lesire to record all features that might be of interest. In his opinion he Paper was especially interesting on account of the candour vith which were disclosed the difficulties that had arisen during he construction of the bridge and the need for the modification f processes which had proved to be unsuccessful. The fact that he scheme had been carried through with a considerable balance n the right side showed the success of the methods finally adopted.

Mr. J. R. DIXON desired only to make a few remarks from the Mr. Dixon. iewpoint of a contractor. The Paper read very smoothly, but it vould be realized that the bridge had not been built without a great nany difficulties having had to be overcome. The work had been f very great interest, and the staff had taken an especial pride in t and had seen that everything had been of the very best. His own irm, as contractors. attached great importance to the maintenance

Mr. Dixon. of close co-operation with consulting engineers, and the cour of the contract under discussion had shown the advantage to l gained by such an attitude. The President had referred in ve generous terms to the manner in which the contractors had carri out the work, and for his part he wished to say how greatly t contractors had appreciated the help and advice which had been freely given them by the engineers.

Mr. Kramer. Mr. J. B. KRAMER remarked that the photo-electric system f automatically and accurately centering the swinging span of t bridge with the roadway had been well explained on p. 744 of t Paper. He did not wish to add anything to that explanation, b merely to emphasize the usefulness and dependability of the phot cell in engineering. He had been responsible for the application the photo-electric equipment to the Kincardine bridge, and wou say that in the hands of an experienced engineer photo-electric c equipment could be applied anywhere with perfect confidence.

The principle of the photo-electric cell was very simple. A ray light, which was itself electro-magnetic in nature, ejected electro from the light-sensitive caesium atoms of the cathode in the ce Those electrons were caught by the anode, so that the light-ener was transformed into electrical energy and a current was establish between the cathode and the anode. That current was very sma but it could readily be magnified by means of electrical valves to any required intensity, so as to work a relay and thus control a kind of apparatus.

(Mr. Kramer then demonstrated the action of a photo-elect control device.)

Since the equipment at Kincardine bridge had been install a peculiar incident had happened; while being swung the brid had stopped when about half-way open. It was then discover that a ray of sunlight had got into a photo-electric cell, having be able to pass the protecting baffles when the bridge was in th particular position. Provision had now been made for disconnecti the photo-cells when the swing-span moved away from the abutme so that the trouble could not recur.

Mr. Deschamps. Mr. JOSEPH DESCHAMPS remarked that when his firm (which h been responsible for the production of the cast-steel rollers used the swing-span) had started on the job they had realized that seri difficulties would be encountered in producing a perfectly sound st casting from the design submitted. The original design had been shown in section in *Figs. 30*. In making the casting, the runners a risers would have had to be arranged as shown dotted, and it wou have been impossible to feed the casting adequately; the pa

iown cross-hatched would thus have been porous and unsound. Mr. Deschamps.
fter considering that design his firm had conferred with the con-
ılting engineers and a slightly different design (*Fig. 31*) had been
rived at, which had satisfactorily solved the problem. His object
mentioning the matter was to point out how helpful it had been to

Figs. 30.

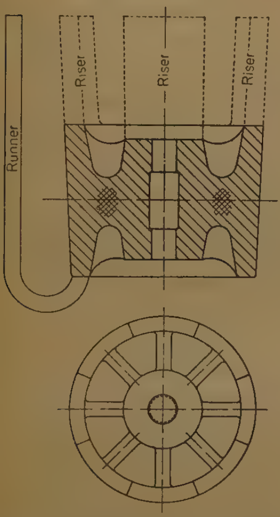
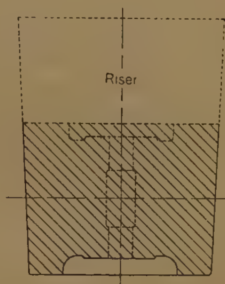


Fig. 31.



e steel-founders to find a considerate firm of consulting engineers
ho were willing to co-operate with them with a view to producing
satisfactory article. The consulting engineers might have insisted
the original design being carried out, in which case bad castings
ould have been produced.

*** Mr. Raymond Carpmael observed that on p. 693 it was stated Mr. Carpmael.
at there were three continuous spans of 62 feet 6 inches over the
& N.E. Railway property, and on p. 694 that " due to the existence
the L. & N.E Railway " a difference of about 22 feet between
ad and rail level was required. It was not apparent from the
neral lay-out why spans of that character had been provided,
d as a material reduction in constructional depth, with compliance
th the Railway Company's requirements for headway, could have
en given with shorter spans, it would be of interest to know whether

*** This and the following contributions were submitted in
riting.—Sec. Inst. C.E.

Mr. Carpmael. they had been provided at the request of the Railway Comp or to conform with the general elevation of the bridge.

Mr. Cook. Mr. F. C. Cook observed that, apart from its engineering inte a feature of the Kincardine bridge project worthy of record was it marked a stage in the advance which had been made in re years in reducing the number of the many gaps which existe the chain of road communications in Great Britain owing to serrated character of the coast-line. The Mersey tunnel, opene 1934, provided a means of crossing the estuary of the Mersey close to its mouth. The new bridge at Kincardine afforded a of several miles in bringing cross-river communication by road ne to the mouth of the estuary of the Forth. Work had been sta upon the new tunnel under the Thames at Dartford, some 15 east of Blackwall tunnel, whilst when the financial position permi he trusted it would be possible for the Government of the day to consideration to the erection of yet another bridge lower dow Forth, as well as bridges across the estuaries of the Humber an Severn.

It was not easy to ensure the pleasing appearance of a stru such as the Kincardine bridge, which was over ½ mile in le with a difference in level between the two shore-ends of no less 15 feet, and incorporating a curve of comparatively sharp ra He was sure that one factor which had contributed to success in direction was the very generous vertical curve which had been throughout. From whatever angle the bridge was viewed, the of that provision was evident, and its value was worth beari mind. Again, there was no attempt at extraneous ornament The design was clearly purposeful and the structure as a whole its setting admirably.

With regard to the use of superelevation, there was someti tendency to regard a bridge as an independent unit and not should be, an essential part of a line of communication, gov by the same requirements as to level and alignment. The c portion of the bridge, which had a radius of about 520 feet, ren superelevation desirable, and that had been achieved, not b addition of filling which would have added materially to the load, but by fabrication of the bridge-members in such a man to provide a superelevation of 16 inches and a widening carriageway of 2 feet 3 inches in the centre of the curve. Th of so fabricating that portion of the superstructure as to give results could not have been easy, but it had secured a m advantage to road-users without in any way detracting fro appearance of the bridge.

The cost of £2 5*s*. per square foot was singularly low, but there was Mr. Cook.
lways some danger in comparing figures of unit cost without due onsideration of the factors involved. There were records of costs f bridge-construction of up to £9 per square foot, but the costs of ridges of similar type might differ substantially, not by reason of ierit or demerit of design, but because of under-water or other ifficulties which might have been met with and which might not be ufficiently appreciated.

Mr. H. G. Lloyd asked what was the grading of the dark sand Mr. Lloyd.
rom the quarry near Denny (preferably in terms of B.S.S. sieves), nd of what shapes were the grains? It was unusual to obtain sands rhich in 3 : 1 portland-cement mortar gave a tensile strength as igh as that of 3 : 1 portland-cement mortar in which standard sand 'as used, and it would be useful if an alternative sand to Leighton uzzard sand could be obtained.

Mr. J. A. Saner wished to support the decision to adopt a swinging Mr. Saner.
oan for navigation rather than a bascule or lifting form of bridge. part from any question of cost or appearance, a swing-bridge could e made to open away from approaching vessels, an important oint in a tideway or flooded river, whereas either of the other rms had to be opened to its maximum height before the vessel ared approach. In many cases that prolonged the delay to road affic.

With regard to the buckle-plate flooring, it would be interesting know what, if any, special care had been taken to ensure an even ad-surface. There was always a tendency for the top dressing to ecome corrugated owing to the material in the trough, being of eater depth than that on the top of the trough, tending to compress ider the road traffic and so cause corrugations.

The load of 3,500 lbs. per linear inch on the rollers appeared heavy. iat might not be of much moment so long as the bridge was operated ily some 500 times in a year, but it had been found necessary in e case at least to put in later an hydraulic ram under the centre to lieve the weight, although the loading had only been about 2,000 lbs. r linear inch. The job had been very expensive, and owing to the ntacts on the centre pillar at Kincardine it would be impossible in at case.

In his opinion solenoid brakes were not suitable for dealing with avy loads, being much too sudden in action, and he had refused have them on movable bridges designed by him.

The effort to turn the Kincardine bridge appeared low, but the efficient of friction of the rollers was somewhat high. That might due to the heavy load per linear inch of roller. The power

Mr. Saner. required per ton per degree of angular movement compa favourably with that required by other bridges.[1]

The Author. The AUTHOR, in reply, desired to express his thanks for the k remarks which had been made about his Paper, especially th expressed by the President and by Sir Charles Bressey.

When writing the Paper he had in view that one of the princi justifications for its submission was that the design, assembly control of a very large swing-span had not been recorded in Institution's publications for many years, and he hoped that fully detailed description of the foundation-difficulties experien the methods adopted for the precise assembly of the structure, the most modern equipment for its control and operation would of value to others, and especially to younger members who m have a similar problem to deal with in future.

With regard to Dr. Anderson's remarks, all of which he gre appreciated, he had found that many Authors of recent Pa had written at equal length, whilst in several instances their Pa had actually exceeded his by amounts of up to 50 per cent. I had sinned, therefore, it had been in good company.

The width of navigation-opening had been increased from 100 to 150 feet in accordance with the decision of the Parliamen Joint Committee. No doubt the Committee had been lar influenced by the gloomy picture painted by representative shipping interests as to the difficulties of negotiating the ope which had indicated conditions of a dense fog and a south-west at the same time !

The preliminary investigations made into the relative costs Scherzer rolling lift bridge and a swing-span with 100-foot ope showed that the former would have cost about £10,000 more. Act definitely required two openings to be provided, so that whe span was increased to 150 feet there was no opportunity po to reconsider the design. The additional 150-foot opening pro by the swing-span was of great value to the navigation autho Mr. Saner had indicateda further advantage of a swing-spa that it was able to turn away from an approaching vessel. was one of the reasons why the span in question had been des to open either clockwise or counter-clockwise.

The sub-division of the bridge costs per square foot, as requ by Dr. Anderson, was as followed :—piled viaduct 17*s*. 6*d*. ; 50 reinforced-concrete spans 20*s*. ; 10-foot spans (average) £1

[1] J. A. Saner, "Swing-Bridges over the Weaver Navigation, with Information about other movable Bridges." Inst. C.E. Selected Engin Paper No. 79 (1929).

wing-span, including machinery, controls, and timber protection- The Author. etty, £6 ; 62-foot 6-inch spans on curve £1 15*s.* ; average £2 5*s.*

A certain amount of scour had occurred at the 100-foot piers on he south side of the river immediately after their construction. The cour had practically ceased thereafter, but it had been considered rudent to stabilize conditions by depositing broken stone around ertain of the piers.

The periods of delays to road-traffic at various opening bridges iven by Mr. Anderson were of great value. The unknown factor n all those cases was the time taken by river-traffic to pass through fter the bridge was open, which was largely affected by tidal and veather conditions as well as by the mentality of the person in harge of the vessel.

With regard to Mr. Carpmael's comment on the span adopted at he L.N.E. Railway, the location of piers Nos. 1 and 2 was determined y definite site-requirements. The railway company had been nost helpful throughout. It was true that if shorter spans could ave been adopted at that place, which had unfortunately been mpracticable, a material reduction in roadway-level would have een possible, though at some sacrifice in the general appearance of he structure.

In reply to Mr. Lloyd, the grading of the Denny sand was as ollowed :—retained on $\frac{3}{16}$-inch sieve, 3 per cent. ; passing $\frac{3}{16}$-inch ieve but retained on 7 sieve, 5 per cent. ; 7–14, 14 per cent. ; 14–25, 8 per cent. ; 25–52, 42 per cent. ; 52–100, 14 per cent. ; passing 100, per cent. The shape of the sand grains followed the general rule, he elongation-ratio of the major and minor diameters being bout 1·5.

No effect of corrugation in the road-surface had been experienced lue to the adoption of buckle plating. The concrete had been very ully reinforced, and that would tend to reduce the action suggested y Mr. Saner.

A very usual formula for the allowable pressure on rollers was 00*d*–250*d*, which gave 4,000 to 5,000 lbs. per linear inch safe oad, whereas 3,500 lbs. per linear inch had been adopted for the ollers on the Kincardine bridge. With regard to the brakes, the voidance of their sudden application was one of the greatest problems o be dealt with. The provision of brakes on a swing-span was a ecessary evil, and the Paper gave particulars of the elaborate recautions which had been taken to prevent their application except vhen the span was at rest.

Mr. Cook's explanation that the generous vertical curve adopted or the bridge had probably the most marked effect on the attractive

The Author. appearance of the elevation was entirely concurred in by the Autho
Whilst he agreed with Mr. Cook that costs per square foot should
viewed with caution, and were mainly of value for approxima
comparisons only, he considered that the detailed statement of co
of the various parts of the structure which he had given in his rep
would provide a very useful comparison when examining costs
other bridges in Great Britain.

*** The Correspondence on the foregoing Paper will be publish
in the Institution Journal for October, 1937.—Sec. Inst. C.E.

JOINT MEETING.

2 March, 1937.

Sir WILLIAM LARKE, K.B.E., in the Chair.

A Joint Meeting, organized by the Institution of Automobile Engineers, was held in the Hall of the Royal Geographical Society, Kensington, S.W.7, with :—

The Diesel Engine Users' Association,
The Institute of Fuel,
The Institute of Marine Engineers,
The Institute of Metals,
The Institute of the Motor Trade,
The Institution of Automobile Engineers,
The Institution of Locomotive Engineers,
The Institution of Petroleum Technologists,
The Junior Institution of Engineers,
The North-East Coast Institution of Engineers,
The Royal Aeronautical Society,
The Iron & Steel Institute,
The Institution of Engineers-in-Charge,

at which a Symposium on Research in relation to the Motor Vehicle was presented.

Section I.—THE MOTOR VEHICLE.

By C. G. Williams, M.Sc.

Abridged Report.[1]

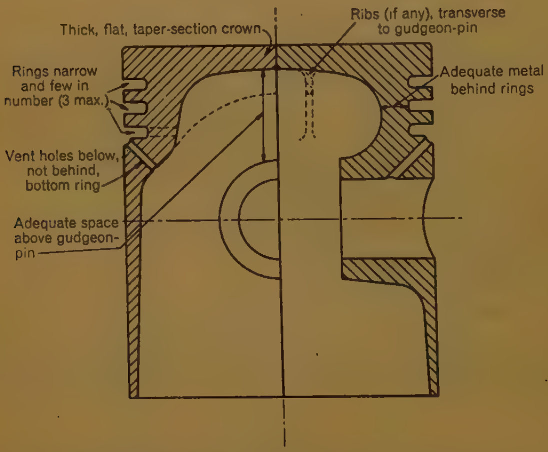
Mr. Williams said that the enormous progress in the design of motor vehicles had been the result of gradual development, which had been assisted by a deeper knowledge of the factors and principles involved, and that it was to research that the automobile engineer looked for light on the functioning of the mechanism in all its details in order that further progress might be made. Research, related directly or indirectly to motor vehicles, was carried out at a diversity

[1] Journal Inst. A.E., vol. v (1937), p. 14 (March 1937). Complete with discussion and written contributions in Proc. Inst. A.E., vol. xxxi, to be issued about Sept. 1937.

of laboratories and works, and, as an example of research by a manufacturer he quoted the cinephotography of flame-propagation in an engine, showing flame-propagation throughout the entire combustion chamber. There had, at the same time, been increasing realization recently of the value of co-operative research on problems which were of common interest to the motor industry, and, in that connexion, he referred to some of the researches carried out by the Institution of Automobile Engineers' Research Department and said that the main trend of modern research, at least on engines, could be summed up in two words " heat " and " wear."

Dealing first with pistons, he observed that work at Manchester University had given data on temperature-distribution as influenced

Fig. 1.

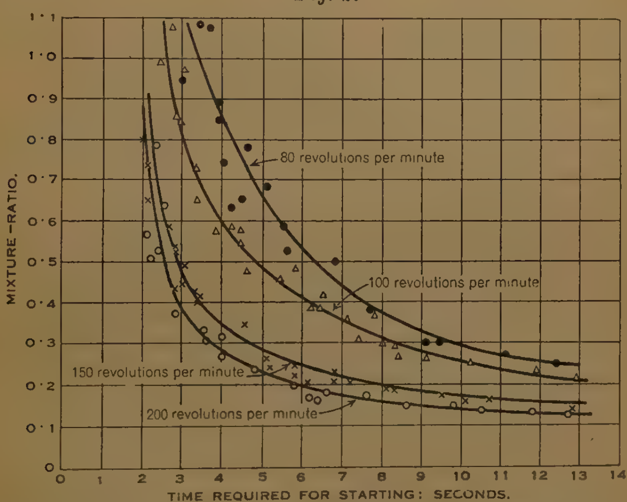


by operating conditions and design features, the principal recommendations resulting from that research, in so far as piston design was concerned, being summarized in *Fig. 1.* Exhaust-valve problems constituted one of the main barriers in the way of increased performance, and nearly every method of improving engine-power almost inevitably had its effect in raising valve-temperatures. Speed had an important effect on temperature and temperatures of over 800° C. could be reached. Tests on the wear of valve-seats indicated a very rapid increase in wear with temperature in an oxidizing atmosphere obtained with a weak mixture, whilst there was negligible wear over almost the whole temperature-range with a rich mixture giving a reducing atmosphere.

A method had recently been developed of measuring big-end temperatures, whilst experiments had been carried out for some

years on special machines in which the durability of bearings could be studied under controlled conditions very similar to those experienced in actual engines. One of the major bearing-troubles experienced was undoubtedly the fatigue-cracking of white metals, and from tests carried out it was found that temperature was a very important factor. In one case, the life of bearings lined with a particular metal was 70 hours at 80° C. and only 15 hours at 140° C. In connexion with cylinder-wear, it was found that engines which were run continuously in a laboratory wore very little as compared with those observed on the road. Further research showed that accelerated wear took place at low temperatures, and led to the establishment of corrosion as a factor of considerable importance

Fig. 2.

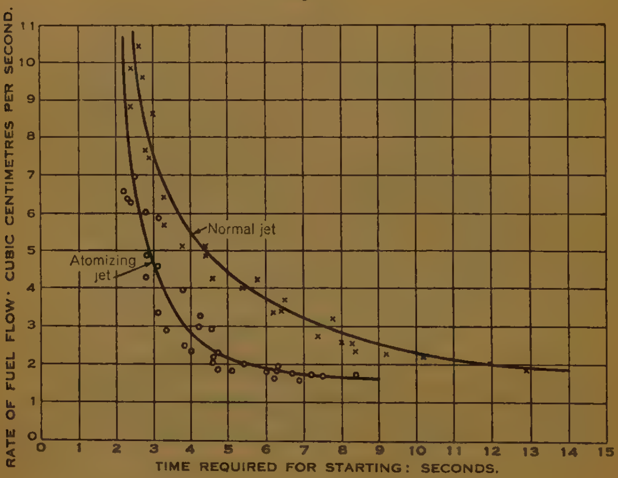


under cold-starting or cold-running conditions. Considering corrosion at low temperatures, the following factors were of importance in reducing cylinder-wear:—(1) plenty of oil immediately on starting; (2) rapid warming up; (3) corrosion-resisting materials; and (4) certain additions to the lubricant. At normal temperatures the following were the factors in reducing abrasion:—(1) clean oil; (2) wide top piston-rings; (3) minimum piston-clearances; and (4) abrasion-resisting materials. There was undoubtedly room for research in the functioning and design of piston-rings, as not only did the design influence wear and oil consumption, but a good deal was heard nowadays about blow-by, ring-flutter, ring-sticking, etc.

There was still room for improvement in the starting performance

of some cars. *Fig. 2* showed some typical graphs indicating how, at any one cranking-speed, the starting time was reduced with increased mixture-strength, whilst increased cranking speed resulted in reduced starting time. *Fig. 3* showed how a special atomizing jet effected a reduction in starting time. Measurements of motoring friction on various engines had shown that the frictional M.E.P. at low temperatures might approach 100 lbs. per square inch, and so the indicated M.E.P. of the engine had to be at least as great for a satisfactory start to be made. The balance between frictional and indicated M.E.P. was the subject of investigation by the Institution of Automobile Engineers' Research Department.

Fig. 3.



Experiments on models had given an exaggerated conception in many cases of the improvement to be expected on the full-size vehicle by streamlining, and it was probably true that the improvement resulting from current attempts amounted to no more than a few per cent. Another factor which had to be investigated was the problem of suspension, in so far as it concerned the comfort of the occupants. It had been found possible to measure quantitatively the amount by which the rear seats were more uncomfortable than the front, and, in the same way, tyres, shock-absorbers, etc., could be studied.

Noise again was a problem that had received increasing attention as designers became noise-conscious. Considerable improvement had been effected by insulation and damping, but the following subjects appeared worthy of further study :—(1) the origin of

drumming noises; (2) the proper application of damping materials to panels; (3) the development and application of sound-absorbing methods capable of dealing with low-frequency sounds; and (4) the elimination of wind noises. With regard to noise from the exhaust, experiments had shown that there were two main frequency-bands to be silenced, one of low frequency, attributed to the resonance chamber formed by the cylinder volume and the valve opening, and the other of high frequency, attributed to the rush of gases through the valve opening. It had been found that certain silencers absorbed high frequencies and not low, and vice versa. Brake-squeak still also caused trouble, and the Institution of Automobile Engineers' Research Department had recently undertaken an investigation to ascertain the origin of squeak.

Section II.—FUELS AND LUBRICANTS.

By F. H. Garner, Ph.D., F.I.C.

Abridged Report.[1]

Dr. Garner said that the changes which had taken place in the character of motor spirits during recent years had been due partly to a demand from the engine designers for fuels modified to overcome certain limitations in engine performance and partly to the increased demand and the necessity for supplying fuels of high anti-knock quality. In so far as volatility was concerned, most of the knowledge of the requirements of motor fuels in spark-ignition engines was based on the work of the Co-operative Fuel Research Committee which was itself a joint committee of the motor and fuel industries. The determination of ease of starting, dependent on the effect of the lubricant and of the fuel (the former being the more important), was a matter of considerable difficulty, because of variations in the design of fuel-induction systems but, as far as petroleum fuels were concerned, it had been found that ease of starting correlated very well with the 10-per-cent.-evaporation point, or the percentage evaporated off at 70° C. in the laboratory distillation-method. It had been found necessary to modify the volatility of motor fuels marketed in different seasons of the year in order to ensure ease of starting in the winter and freedom from the difficulties attendant on the use of too volatile a fuel in the summer.

For satisfactory warming-up it was important that, as soon as the engine was started, the fuel should be supplied in a condition so that it could readily be burnt and enable the engine to develop its full power. The experimental work of the C.F.R. Committee showed that maximum acceleration could be obtained with any fuel, but if the fuel was of lower volatility a richer mixture was required. The design of the induction system appeared to be the deciding factor rather than the fuel, but subject to that consideration, the 50 per cent. point in the distillation-range of the motor fuel appeared to be an index of the acceleration and warming-up characteristics.

Dr. Garner also referred to vapour lock, of which there were two forms: one in idling or light traffic, shown by stalling followed by difficulty in starting, and the other at higher speeds, shown by loss of power similar to that felt in momentarily running out of fuel. That problem was much more important in the export market, where a greater range of temperatures would be found, and it was there-

[1] Journal Inst. A.E., vol. v (1937), p. 33 (March 1937). Complete with discussion and written contributions in Proc. Inst. A.E., vol. xxxi, to be issued about Sept. 1937.

fore important to ensure that all parts of the fuel-system were kept at as low a temperature as possible.

After mentioning that at the present time dilution of the crank-case-oil could not be considered of serious importance, he dealt with fuel-consumption, and pointed out that as a result of improvements in the design of the fuel-system a marked improvement had occurred in connexion with ease of starting, vapour lock, acceleration and dilution, owing largely to the use of easy-starting fuels of controlled volatility. That, however, had not been accompanied by a decrease in fuel-consumption, mainly because the fuel/air mixture supplied to the engine was richer overall than that theoretically required for maximum economy.

The importance from the point of view of efficiency of the anti-knock qualities of petrol had been demonstrated, and the introduction of lead tetraethyl had been of the greatest importance in that respect. The refiner had also developed methods of cracking to give fuels of high octane number, including re-forming, which consisted of cracking petroleum fractions to more volatile products of good anti-knock quality. A more recent development was the introduction of polymerization, in which gases from cracking-operations were transformed into liquid products of high octane number.

Brief reference was made to the rating of diesel fuels based on the reference fuels cetene or cetane, and alpha-methyl naphthalene. The question of fuel-economy was one of great importance with diesel engines, although no gain in efficiency was obtained by the supply of fuels of better than the necessary quality. It would, however, be possible to use lower compression-ratios and therefore obtain some saving in weight by the use of such fuels.

It was pointed out that it was only by the development of new methods of refining that motor lubricating oils had become available with the relatively low change in viscosity with temperature necessary for easy starting. The term "viscosity-index" had been widely adopted as a method of expressing the viscosity—temperature relationship of lubricating oils. Since oils of high viscosity-index had correspondingly high viscosities at high temperatures, the grade of oil used could be of lower viscosity at ordinary temperatures, and there was no doubt that the time was overdue for the use of lower-viscosity oils in Great Britain for the lubrication of motor-car engines. The main causes of high oil-consumption, apart from the mechanical condition of the engine, were high engine-speeds and oil leaks, in both of which the viscosity of the oil played an important part.

Section III.—MATERIALS—WITH SPECIAL REFERENCE TO STEEL.

By T. Swinden, D.Met.

Abridged Report.[1]

Dr. Swinden said that in the design of materials for automobile construction there was continual striving for improved performance accompanied by the most strenuous efforts to reduce costs. In dealing with the subject, he had used the provisional specifications issued by the Society of Motor Manufacturers and Traders as a background on which to draw attention to the trend of development in the types of steel used in the more important parts of automobile construction. He felt, personally, that the Society and the Institution of Automobile Engineers were well advised in concentrating on the issue of a set of specifications drawn up to represent the requirements of the automobile industry. It was clearly in the interests of the steelmakers to collaborate to the full extent in any effort made towards standardization which represented progress in the establishment of specifications covering the steels which the industry desired to use. Whether standardization would result in economy of production, or not, clearly had to depend on whether the steels which were ultimately standardized represented the most economical steels for the purpose. He was not himself convinced that unification of standards covering the supply of steels to the automobile and aircraft industries was necessarily sound or the best procedure, as the requirements for the two industries were not necessarily the same.

Taking first of all that most important member, the crankshaft, attention was directed in some detail to the manganese-molybdenum and chromium-molybdenum qualities of steel used. Some notes were included concerning various methods of surface-hardening, and the dangers attached to cold straightening were illustrated by some tests recently carried out.

After a note on the inter-penetration of non-ferrous metals, the properties of accepted types of valve-steels were enumerated. The subject of gear-steels was then dealt with, under which the

[1] Journal Inst. A.E., vol. v (1937), p. 53 (March 1937). Complete with discussion and written contributions in Proc. Inst. A.E., vol. xxxi, to be issued about Sept. 1937.

relative merits of air-hardening, case-hardening and direct-hardening steels were discussed, with particular reference to the various methods of case-hardening and to the importance of the control of grain size in determining the quality of the steel and in permitting the use of lower-alloyed steels for many parts where the most expensive of highly-alloyed steels were formerly considered to be indispensable. Reference was made to the steels used for shafts and similar purposes, and opportunity was taken to draw particular attention to the properties and usefulness of the newer manganese and manganese-molybdenum and chromium-molybdenum steels for many parts of the automobile, although the service to the automobile industry of the well-known nickel, nickel-chromium and nickel-chromium-molybdenum steels was fully appreciated.

Reference was also made to the subject of springs, note being taken of the coating of spring-plates with the dual object of reducing corrosion and interleaf friction.

The concluding section dealt with steels for chassis and body construction, attention being drawn to certain lines of development of research, particularly in reference to body sheets, higher-tensile steels suitable for welding, the possibilities of stainless-clad mild steel and the development of rapid-machining steel having very good mechanical properties and reliability. The Paper closed with a plea for a more detailed study of the significance of the mechanical tests which were being applied to steel in assessing its suitability for service in various forms.

Paper No. 5096.

"The North Bihar Group of the Rivers and Waterways of the Gangetic Plain before and after the Earthquake of the 15th January, 1934."

By FREDERICK CHARLES TEMPLE, C.I.E., M. Inst. C.E.

(*Ordered by the Council to be published in abstract form.*)[1]

THE rivers of the Gangetic plain are said to be the most important factors in the development of the plain; although they can be temporarily curbed, go their own way in the end. The Ganges and its many tributaries traverse the plain from end to end and of them the Ganges and its northern tributaries have characteristics in common. Rising in gorges in the Himalaya or in swamps at their foot, they flow south or west of south as far as they can and then turn eastwards, always tending to prolong the southerly movement as far as possible until they come up against the hard northern edge of the Central Indian plateau. The plain is being raised by detritus brought down by the rivers from the hills. Having once been lowest next the hills, its slope is now steepest there, flattening out more and more towards the south.

The Paper describes the characteristics of the rivers of North Bihar, which form a typical group, and from whose past history some idea of their probable future can be obtained. They comprise "live" rivers which run all the year, and channels which are now merely spillways and only run in the rains. The annual rainfall in Bihar is about 50 inches, of which 80 per cent. falls in $3\frac{1}{2}$ months. As the hot weather and rains come on, the snow-fed rivers rise first, then the other "live" rivers, and finally water flows in the old channels.

The rivers often change their courses after a flood. In high floods the water spreads out over the country, moving with the greatest velocity and highest surface-levels at the river beds. The silt-carrying rivers are land-building rivers. They deposit the silt on either side and on their beds and so build up ridges with their beds as grooves in the crown of the ridge, until the ridges are so high that the rivers fall off to one side or the other and start building in another

[1] Photostat copies of the full Paper can be obtained on loan from the Loan Library of the Institution; a limited number of photostat copies are also available, for retention by members, on application to the Secretary.

place. If not interfered with, the land is raised fairly evenly, but embankments are often made in order to avoid the immediate inconvenience and loss due to flooding.

Another cause for rivers changing their courses is said to be the frequent disturbance of the delicate balance between the velocity of the river and the soil through which it flows. There is only one velocity for each river, the "critical" velocity, at which the river can flow through alluvium without either scouring or silting its channel.

Reference is made to the policy followed until about 30 years ago of building embankments to protect favoured localities from flood, which gave rise in certain cases to waterlogging of these localities. The effect of earthquakes in sometimes raising the beds or contracting the channels of rivers or causing tracts of country to sink is also considered. Based on these general conditions the North Bihar rivers are discussed individually from west to east, and certain conclusions arrived at which show that it would be best for the country to be evenly raised by the rivers going into one low place after another, the rivers being guided by digging channels rather than by erecting embankments. An exception should, however, be made with the river Bagmati which, until it has finished its land-building across the middle of Muzaffarpur district, should be persuaded to keep out of the river Burhi Gandak. This could best be done by maintaining an embankment on the divide between the two rivers.

ENGINEERING RESEARCH.

DETERIORATION OF STRUCTURES IN SEA-WATER.

The Sixteenth (Interim) Report[1] of the Committee of The Institution of Civil Engineers appointed to investigate the Deterioration of Structures of Timber, Metal and Concrete Exposed to the Action of Sea Water has recently been published. The Fifteenth Report, which appeared a year ago, and a brief note on which appeared in the April, 1936, Journal,[2] gave a complete general account of the whole of the investigations up to that time. The present Report gives a detailed account of the subsequent progress of the investigations.

A report is given by Professor George Barger on the examination of timber specimens which had been impregnated with creosote or fuel oil with or without the addition of poisons, and which had been subsequently exposed for long periods in various parts of the world. The variability in the behaviour of specimens makes it difficult to draw definite conclusions regarding the effectiveness of particular treatments. The results indicate, however, the value of treatment with creosote with or without added poison as a protection against *teredo*. As regards attack by *limnoria* the experiments are not so conclusive. Treatment with naphthalene or naphthalene mixtures would appear to be ineffective. A series of posts are being exposed at Colombo, some of which are incised at two different spacings whilst some are unincised, the intention being to determine the effect of incision in securing better penetration of creosote with consequent increase of resistance to attack. It is too soon to draw conclusions from these tests, both incised and unincised specimens being in good condition at the time of the report. Various untreated native timbers on the Gold Coast are noted by Professor S. M. Dixon to be very resistant to attack by marine borers.

Progress reports which had been received from Auckland, Halifax, Colombo, and Plymouth described the condition of the 15-year specimens of iron and steel which were still under exposure.

An analysis made by Dr. J. Newton Friend of various commercial

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[1] The Report is published for The Institution of Civil Engineers by His Majesty's Stationery Office under the authority of the Department of Scientific and Industrial Research, and copies, price 6*d.* may be obtained from H.M. Stationery Office, or through any bookseller.

[2] Journal Inst. C.E., vol. 2 (1935–36), p. 585. (April, 1936.)

galvanized articles showed a wide variability in the amount of coating. It is shown that the galvanized test-plates used by the Committee were unusually heavily galvanized, a feature which would explain their high resistance to corrosion.

A report is included by Dr. R. E. Stradling on the deterioration of reinforced-concrete test-pieces in sea-water, being a description of experiments in progress at the Building Research Station, Sheerness, and on the Gold Coast. The experiments at the Building Research Station indicate the need for adequate cover over the reinforcement, the advantage of a rich concrete mixture, and the superior resistance of concrete of dry consistency. Whilst further specimens have shown signs of cracking, there is as yet no case of cracking with 2-inch cover and medium mix of dry consistency. Inconsistent results have been obtained with high-aluminous cements and with concrete containing artificial pozzuolanas, and further work is being carried out thereon.

THE INSTITUTION RESEARCH COMMITTEE.

Joint Committee on Simply-Supported Steel Bridges.

The formation of this Committee jointly with the Institution of Structural Engineers was announced in the October, 1936, Journal.[1] The personnel of the Committee has been increased by the appointment of Mr. Ralph Freeman in that month. Since then meetings have been held at approximately fortnightly intervals and good progress has been made in the preliminary consideration of a code of practice and design.

In attempting to formulate rules for the calculation of wind loads the Committee came to the conclusion that the present state of knowledge on this subject is not sufficiently definite to make it possible to prescribe allowances for wind forces with any degree of confidence. Information on wind-pressures is still largely based upon the experimental work of the late Sir Thomas Stanton,[2] although recent researches have been conducted in America, in Germany, and at the National Physical Laboratory. Further research, however,

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[1] Journal Inst. C.E., vol. 3 (1935–36), p. 601. (October, 1936.)

[2] "On the Resistance of Plane Surfaces in a Uniform Current of Air." Minutes of Proceedings Inst. C.E., vol. clvi. (1903–4, Part II), p. 78.

"Experiments on Wind-Pressure." *Ibid.*, vol. clxxi (1908–9, Part I), p. 175.

"Report on the Measurement of the Pressure of the Wind on Structures." *Ibid.*, vol. ccxix (1924–5, Part I), p. 125.

is necessary before wind-pressure allowances on bridges can be formulated with any degree of confidence. In particular, information is required concerning the effects of the shape of the cross-section of members, the shielding of leeward girders, the bridge-flooring, and the obliquity of the wind.

Research on Wind-Pressure on Bridges.

The Institutions of Civil and Structural Engineers have accordingly decided to institute a research into wind-pressures on simply-supported bridges. The Department of Scientific and Industrial Research has agreed to assist, both financially and by the carrying-out of tests on bridge-models in the 7-foot-diameter wind-tunnel at the National Physical Laboratory. A preliminary scheme of research has been agreed upon, and the preparation of test-models is to be commenced forthwith.

It is hoped to investigate both plate- and lattice-girder construction. In the case of the latter the research will be directed in the first place to an investigation of the possibility of replacing "true-to-scale" model-members by conventional rectangular members of the same size. The effects of the transparency-ratio, the spacing of main girders, the type of floor-construction, the presence of a train on the bridge, and other factors will then be investigated, using models with simple conventional members.

REPORT OF THE WATER POLLUTION RESEARCH BOARD FOR THE YEAR ENDED 30TH JUNE, 1936.

The urgency of the problem of water pollution is emphasized in view of the increasing demand for large volumes of water of good chemical and bacteriological quality, in addition to the increasing consumption of water of the highest quality.

Particular attention is drawn to the work of purification of milk-factory effluents. This is a problem of great urgency and is being investigated in co-operation with the Milk Marketing Board and the Scottish Milk Marketing Board. Laboratory and large-scale experiments have shown that it is practicable to purify milk-washings by biological oxidation in percolating filters or by the activated-sludge process. The economics of such purification are being studied. The investigation has brought to light the possibility of effecting a direct saving by the elimination of waste of milk and its products.

The purification of sewage by means of activated sludge appears

to be the result of both physical and biochemical or biological changes. A study has been made of the stage of biochemical oxidation.

Experiments have been continued into the process of water-softening using base-exchange zeolites. To avoid the importation of foreign materials, experiments are being made with the object of preparing satisfactory base-exchange materials from British clays. Investigation of the base-exchange and acid-exchange properties of synthetic resins prepared from various phenolic substances and aromatic bases by condensation under different conditions with formaldehyde has been continued. Widespread interest has been aroused in this discovery and several firms are investigating the commercial possibilities of the processes. An improved method has been evolved for determining the true concentration of lead in household drinking water over a period of several weeks.

An investigation on the river Mersey with the object of determining the effect of the discharge of crude sewage on the nature and amount of the solid matter deposited in the estuary has been continued. In addition, a study has been made of the changes in the estuary during the past 75 years, with a view to obtaining evidence of any changes which may affect the removal of material deposited therein and in Liverpool Bay. Deposits from this and other estuaries have been examined by various methods and the effects of different conditions on the rate of sedimentation of mud and silt from suspension have been studied.

Useful work by the Liquor Effluents and Ammonia Committee of the Institution of Gas Engineers has been noted. The Fresh-water Biological Association of the British Empire has made further progress in the study of the biology, chemistry, and physics of relatively unpolluted waters.

THE BRITISH ELECTRICAL AND ALLIED INDUSTRIES RESEARCH ASSOCIATION.

The report on the work done during the year ending on the 30th September, 1936, presented at the annual meeting of the British Electrical and Allied Industries Research Association, shows an expanding field of activity.

The value of the researches is emphasized by the work on steam turbines, boilers, and pipes, in which the return to industry arising therefrom exceeded the total income of the Association from all sources. In this connexion valuable knowledge has been gained of creep and corrosion of steel at high temperatures, and the design of

pipe-flanges for high temperatures has been studied in co-operation with the Institution of Mechanical Engineers. As a result of these and other researches it has been possible to effect considerable improvement in the efficiency of steam generating-stations.

Research has been continued on behalf of the Central Electricity Board into surges and allied phenomena, much of the work being carried out in the new high-voltage laboratory at the National Physical Laboratory. Researches on the phenomena of circuit-breaking have been carried out with special generating plant in the new auxiliary laboratory, which was opened at the beginning of the year. The practice and principles of earthing have been investigated with a view to the preparation of British Standard Specifications on the subject, and it has been possible to give valuable assistance in connexion with interference with communication circuits. A section has been formed to deal with safety problems, and work on fire extinction in relation to the particular requirements of the electrical industry is now being planned. A large amount of work has been done on the fundamental properties of dielectrics and problems in connexion with high-tension direct current are receiving attention. The nature of electrical breakdown has been further studied and proposals have been prepared for British Standard Specifications for insulating materials, particularly in respect of brittleness and ageing tests. A study of flameproof switchgear has been extended to cover industrial requirements, other than mining work, where inflammable gases may be present. More than one solution has been found to the problem of producing a more reliable bottom bearing for integrating wattmeters, so as to reduce maintenance costs and loss of revenue from slow-running meters.

Special methods required for the study of transformer-noise at its source have been evolved, and the general study of noise in relation to buildings is in progress. The best method of assessing the quality of electricity-supply in respect of voltage-variation is being investigated. In the field of interference with communication circuits and the problems arising from developments in the radio industry and television, attention has been concentrated on short-wave work, on mercury-arc rectifiers as a source of trouble, and on associated earthing-problems.

NOTES ON RESEARCH PUBLICATIONS.

Measurement.

The application of photography to surveying is described in *J. Inst. Mun. & County Engineers*, **63,** *1181,* and on *p. 1229* the principles of surveying from vertical air photographs are enunciated. A statistical investigation of rainfall intensities and frequencies based on U.S. Water Bureau publications and limited to excessive storms of 2 hours' duration or less is given in *Proc. Am. Soc. C.E.*, **63,** *225.*

Engineering Materials : Properties and Testing.

A summary of literature on the mechanical properties and elasticity of solid bodies is given in *Die Physik,* **4,** *131.* Tests on shearing phenomena at high pressures, particularly in inorganic compounds, are described in *Proc. Am. Acad. Arts & Sciences*, **71,** *387.*

Timber.

Tensile-strength tests perpendicular to the grain of timber under various temperature and moisture conditions are summarized in *Council for Scientific & Industrial Research, Australia, J.*, **9,** *265.*

Cement and Concrete.

A Paper on strength tests for cement, containing proposals for standard specification, giving the result of work at the Building Research Station, is given in *Structural Engineer,* **15,** *50.* A statistical study of tensile tests extending back to 1927 on one brand of Portland cement is given in *Univ. Maine Technology Expt. Stn. Paper No. 18.* In *U.S. Bur. Stand. J. Research*, **17,** *895* (*Eng. Abs.* **73,** 11) differences in limes and their mortars are discussed. An article on grading and workability, describing the compacting factor evolved at the Building Research Station, is given in *J. Am. Conc. Inst.*, **8,** *319.* In *J. Boston Soc. C.E.*, **24** (*1*) *28* the practical application of catalysis

The figure in heavy type is the number of the Volume ; that in brackets the number of the Part ; and that in italic type the number of the Page ; in references to "Engineering Abstracts," the number of the Abstract is given.

and dispersion to cement and concrete is discussed following investigations of the catalytic effect of the admixture of various salts on the setting and hardening and other properties of concrete. The thermal expansion of concrete is dealt with in *Beton und Eisen*, **35**, *401* (*Eng. Abs.* **73,** 12) and tests on shrinkage of concrete are described in *Ohio State Univ. Eng. Expt. Stn. News*, **8** (*4*) *6*. The presence of reinforcement in drilled concrete cores is shown in *J. Roy. Tech. Coll. Glasgow*, **4** (*1*) *135*, to have no appreciable effect upon crushing strength. Tests on 10-year old concrete are discussed in *Univ. Wisconsin, Eng. Expt. Stn. Reprint No. 48*. Measurements of volume-change in concrete structures are given in *Betong*, *1936* (*2*) *76*. Experiments with concrete admixtures (mortar-waterproofers) are described in *Zement*, **25,** *459*. In *J. Am. Conc. Inst.*, **8,** *339*, is given a study of sub-aqueous concrete.

Metals.

Tension and notched-bar tests on a nickel-molybdenum alloy steel are discussed in *J. Roy. Tech. College, Glasgow*, **4** (*1*) *1*. The behaviour of steel at high temperatures under alternating tensile stress is dealt with in *Kaiser-Wilhelm Inst. Eisenforsch.* **18,** *163*. Data on the rate of growth of fatigue cracks are given in *J. Applied Mechanics*, *1936*, **3** (*1*) *A23*. A book "The Corrosion of Metals," by C. Grard, *Paris*, *1936*, has been noted. The influence of impurities on the properties of lead is discussed in *Proc. Australasian Inst. Min. Met.*, *1936* (*101*), *33* and *57*. In a series of articles on the creep of lead and lead alloys, the creep of virgin lead is dealt with in *J. Inst. Metals*, **3,** *623*.

Other Materials.

The mechanical properties of stone as dependent on particle-size and pore-volume, considered in relation to the mechanical properties of insulating materials, slag blocks, and similar artificial granular products, are discussed in *Stein-Ind.*, **31,** *313*. Standard methods of test for absorption and apparent specific gravity of natural building stone are given in *Am. Soc. Testing Materials, Standards*, *1936*, (*2*), *144*, and on *p. 147* is given a standard method of flexure testing of natural building stone. The results of an investigation into the thermal expansion of typical American rocks are contained in *Iowa Eng. Expt. Stn. Bull. No. 128*. Research into the use of asphalt mastic for roofing carried out at the Building Research Station is described in its *Special Report No. 25*.

Engineering Materials: Production, Manufacture, and Preservation.

Methods of electric curing of concrete products are reviewed in *Concrete*, **144** (*11*) *21*. The waterproofing of concrete by bitumen is discussed in *Stroitelnaya Promishlennost*, **14** (*18*) *10* (*Eng. Abs.* **73,** 57). The adherence of sprayed metal coatings is discussed in *Zeit. Metall.*, **29,** *63*. In *Récherches et Inventions*, **17,** *189*, is a contribution to the study of the mechanical properties of paint- and varnish-films, in which proposed methods of test are described. Tests of stone preservatives on actual structures extending over the last 10 years are given in *Bautenschutz*, **7** (*7*) *73*.

Structures.

Mass Structures.

The relation between porosity and pore water-pressure in clays is considered in *Bauing.*, **17,** *559* (*Eng. Abs.* **73,** 10). The grouting of contraction joints at the Boulder dam is described in *Civ. Eng.*, **7,** *126* (*Eng. Abs.* **73,** 55). The composition of the Portland-pozzuolana cement used in the Bonneville spillway dam is given in *J. Am. Conc. Inst.*, **8,** *183*, and on *p. 327* experiments carried out at the Massachusetts Institute of Technology on the drying shrinkage of large concrete members are described.

Framed Structures.

Buckling in elastic materials is dealt with in *Annales des Ponts et Chaussées*, **106**-*ii*, *443* (*Eng. Abs.* **73,** 41). A mathematical analysis of the axial bending stresses in thin cylindrical shells with flat and spherical ends is given in *J. Roy. Tech. College, Glasgow*, **4** (*1*) *85*, and in the same journal, *p. 121*, is an article on stress-distribution in irregular sections using photo-elastic models. The bursting of a steel flask under test is described in *Assoc. française de Propriétaires d'Appareils à Vapeur*, **17,** *243* (*Eng. Abs.* **73,** 18). The stability of the corner-posts of lattice masts is discussed in *Bauing.*, **17,** *557* (*Eng. Abs.* **73,** 12). A diagram for the calculation of tubular cantilevers with ends constrained to circular form is given in *J. Franklin Inst.*, **222,** *737*. A specification for the design, fabrication, and erection of structural steel for buildings, revised 1936, has been issued by the American Institute of Steel Construction. The design of steel wind-

bents for tall buildings based on the results of tests carried out at the Ohio State University is discussed in its *Eng. Expt. Stn. Bull. 93.* The existing knowledge of wind pressures on structures is reviewed in *Ricerche di Ingegneria*, **4,** *105.* A book by R. Saliger and E. Bittner (*Vienna, 1936*): "Tests on reinforced-concrete beams under static and impact loads," describes tests carried out at the *Technische Hochschule*, Vienna. An investigation on the effect of stiffness of floors on the horizontal vibrations of a framed structure is described in *Bull. Earthquake Research Institute, Tokyo Imperial University*, **14,** *367*, and the principles of design of earthquake-resistant structures in reinforced concrete are enunciated in *J. Am. Conc. Inst.*, **8,** *223.*

Transformation, Transmission and Distribution of Energy.

The production of power from sewer-gas is discussed in *Archiv Wärme*, **18,** *53.* Information regarding the explosion of a locomotive boiler is contained in *Rev. Gén. Chemins de Fer*, **56**-*i*, *106.* Experience in connexion with salt-deposits on the blading of steam turbines and their removal by washing with steam is given in *Wärme*, **60,** *117* (*Eng. Abs.*, **73,** 78). The method of fuel-injection for isobaric combustion in a compression-ignition engine is discussed in *Comptes Rendus*, **204,** *556.* The performance of coil-ignition systems, with particular reference to double contact-breakers and the effects of variation of the period of open circuit, is described in *J. Inst. Elec. Engineers*, **80,** *329.*

A theoretical and experimental analysis of the effect of impulse voltages on transformer windings, with a view to their testing, is given in *J. Inst. Elec. Engineers*, **80,** *117.* A new aluminium cable for large currents is described in *Elek. Zeit.*, **58,** *123* (*Eng. Abs.* **73,** 91).

Mechanical Processes, Appliances, and Apparatus.

Welding procedure in relation to physical properties is discussed in *Welding Industry*, **5** (2) *68*, and calculations of economy, soundness, avoidance of distortion and residual stress, and design for static and alternating stresses are dealt with. Tests on welded plates with sheared and flame-cut edges are compared in *Schiffbau*, **38,** *34* (*Eng. Abs.* **73,** 107). A study of the acetylene welding process so as to avoid the formation of oxide in the welding of light-walled tubing is given in *J. Eng. Institute of Canada, Aeron. Section, Reprint No. 7, Dec. 1936, p. 28.* The welding of pressure-vessels is discussed in

Am. Soc. Nav. E., **48,** *498* (*Eng. Abs.* **73,** 105). Nickel-copper high-strength steels for welded construction are discussed in *The Welding Journal*, **16** (*2*) (*Welding Research Supplement*), *p. 2*, and on *p. 7* is given a review of the literature on the welding of copper and its alloys. Laboratory corrosion tests of welded low-carbon stainless steel are given in *U.S. Nat. Bur. Stand. J. Research*, **18,** *69.*

A symposium of Papers on corrosion in the refrigerating industry is given in *Proc. British Assoc. Refrigeration*, **33** (*1*) *35.*

Specialized Engineering Practice.

Transport.

Some important considerations in highway design are recorded in *Civ. Engineering*, **7** (*1*) *1*, and on *p. 5* are instanced recent practical applications of soil mechanics to highway construction, with special reference to the work of the Ohio State Testing Laboratory, whilst on *p. 17* the weathering of asphalt pavements is discussed. A Paper on the shape of crushed stone and gravel and its measurement, describing work done at the Road Research Laboratory, is given in *Chemistry and Industry*, **56,** *206* and on *p. 248*, is a Paper discussing particle-shape and surface characteristics of aggregates. Experiments on the development of special cements for road construction are discussed in *Zement*, **25,** *791.* The structural design of concrete pavements is dealt with in *Public Roads*, **17,** *175* (*Eng. Abs.* **73,** 120). The design requirements of a road for present-day traffic are explained in *Univ. Illinois Eng. Expt. Stn. Circ.*, **27,** *107* (*Eng. Abs.* **73,** 121). A book has been published : " Experiments with concrete surfacings for the German motor roads carried out in the years 1934–5," by O. Graf, *Berlin, 1936.* A publication : Note on the efficient vibration of concrete, particularly in modern concrete road construction, by H. Dienst, *Technische Hochschule*, Berlin, has appeared, *Berlin 1936.* The following researches on motor-vehicles have been noted : *J. Inst. Aut. Engineers*, **5** (*6*), *p. 14*, Symposium on research in relation to the motor-vehicle : Section (i), The motor vehicle ; Section (ii), Fuels and lubricants ; Section (iii), Materials—with special reference to steel ; and in the same journal, *p. 78*, The silencing of gas-noises for all classes of vehicles ; *p. 96*, Properties of some materials for cast crankshafts, with special reference to combined stresses.

An article on the strength of welded joints of various types of construction is given in *Quarterly Technical Bull. of the Railway Board, India*, **4** (*44*) *17.*

Wave-action on vertical breakwaters is discussed in *Sci. et Ind.*

(*Travaux*), **21,** *13* (*Eng. Abs.* **73**, 128). In *J. Roy. Tech. Coll., Glasgow*, **4,** (*1*) *194* the effect of length of hull on the effective horse-power of ships is considered.

A Paper on aerodynamic and structural features of tapered wings is given in *J. Roy. Aero. Soc.*, **41,** *162.* A comparison of tests on a model split-flap aerofoil in an open-jet wind-tunnel is made with the full-scale phenomena in *J. Roy. Tech. Coll., Glasgow*, **4** (*1*) *94.* The following Aeronautical Research Committee *Reports and Memoranda* have been noted; *No. 1710*, Tests on two streamline bodies in the compressed-air tunnel; *No. 1726*, An analytical comparison of model and full-scale spinning experiments on a Bristol fighter; *No. 1730*, The flight of a helicopter; *No. 1733*, A new form of dashpot with a large range of damping; *No. 1745*, On the static pressure in fully-developed turbulent flow; *No. 1747*, Further measurements of ground interference on the lift of a Southampton flying boat; *No. 1750*, *Reports and Memoranda* published between 1st April, 1935, and 30th November, 1936; *No. 1752*, Calibration of standard pitot-static heads in the high-speed tunnel. Apparatus for recording the deformation and vibrations of an aircraft propeller during flight is described in *Comptes Rendus*, **204,** *479.* The following researches have been noted in *Luftfahrt.* **14**; *p. 45*, Oblique and swept-back aeroplane wings; *p. 55*, Reduction of drag on wings at supersonic speed by biplane arrangement; *p. 63*, Stresses in ring-frames fitted for the introduction of longitudinal forces in cylindrical and conical shells; *p. 71*, Pressure-distribution measurements in an aeroplane-wing with landing-flap section; and *p. 86*, Stability of groups of plane framework members.

The following Reports of the U.S. National Advisory Committee for Aeronautics have also been noted: *No. 556*, Further studies of flame-movement and pressure-development in an engine cylinder; *No. 557*, Preliminary tests in the N.A.C.A. free-spinning wind-tunnel; *No. 558*, Turbulence factors of N.A.C.A. wind-tunnels as determined by sphere tests; *No. 559*, The forces and moments acting on parts of the XN2Y–1 airplane during spins; *No. 560*, A simplified application of the method of operators to the calculation of disturbed motions of an airplane; *No. 561*, Effect of nozzle design on fuel-spray and flame-formation in a high-speed compression-ignition engine; *No. 562*, Air-flow in the boundary layer near a plate; *No. 563*, Calculated and measured pressure-distributions over the midspan section of the N.A.C.A. *4412* airfoil; *No. 564*, Tests of a wing-nacelle-propeller combination at several pitch-settings up to 42 deg.; *No. 565*, Measurements of fuel-distribution within sprays for fuel-injection engines; *No. 566*, Ground-handling forces on a 1/40-scale model of the U.S. airship "Akron"; *No. 578*, Flight

measurements of the dynamic longitudinal stability of several airplanes and a correlation of the measurements with pilots' observations of handling characteristics ; *No. 580.* Heat-transfer to fuel-sprays injected into heated gases.

Water-Supply and Sewage-Disposal.

Laboratory experiments over a wide range of Reynolds numbers from 500 to 600,000 on pressure losses for fluid flow in curved pipes are described in *U.S. Nat. Bur. Stand. J. Research*, **18,** *89.* Tests on wave-propagation in the canal leading to a hydro-electric station are given in *Gidrotechnicheskoe Stroitelstvo, 1936*, (**12**) *26.* (*Eng. Abs.* **73**, 151). The ozone treatment of drinking water is discussed in *Gas und Wasserfach*, **80,** *180.* An article on the destructive action of domestic sewage on metal is contained in *Gesund Ing.*, **60,** *69* (*Eng. Abs.* **73**, 162).

Mining.

Notes on the use of hydraulic coal-bursters instead of shot-firing are given in *Trans. Inst. Mining Engineers*, **92,** *300.* In *J. Roy. Tech. Coll., Glasgow*, **4** (*1*) *178* are described experiments to determine the ease of wetting of mine dusts with water and various solutions. In *Safety in Mines Research Board Paper No. 97*, the effect of fibre cores on internal corrosion in colliery winding-ropes is discussed and it is shown that corrosion may be caused by acid liberated as a result of bacterial action.

Lighting, Heating, and Acoustics.

The daylight illumination necessary for clerical work is discussed in *Illumination Research Technical Paper 19.* Tables for the estimation of solar heat transmitted through walls and roofs are given in *Heating and Ventilating*, **33,** *37.* Methods of determining the resonance frequencies of vibration of a building partition at audio-frequencies are given in *J. Phys. Soc., Proc.*, **48,** *914.*

Telegraphy and Telephony.

An article on recent developments in telegraph transmission and their application to the British telegraph services is given in *J. Inst. Elec. Engineers*, **80,** *237*, and on *p. 286* ultra-short-wave refraction and diffraction is described.

Miscellaneous.

In *Proc. Roy. Soc., Series A*, **158,** the following papers have been noted: *p. 499,* Mechanism of the production of small eddies from large ones; *p. 522,* Motion of an infinite elliptic cylinder in fluids with constant vorticity.

NOTE.

The Institution as a body is not responsible either for the statements made, or for the opinions expressed, in the Papers published.

OBITUARY.

Sir JOHN AUDLEY FREDERICK ASPINALL was born on the 25th August, 1851, in Liverpool and died on the 19th January, 1937, at Woking. He was educated at Beaumont College and after leaving school became a pupil at the Crewe locomotive works of the London and North Western Railway, first under John Ramsbottom and subsequently under F. W. Webb. At the end of his pupilage he was appointed Assistant Manager at the Crewe steelworks and remained there for some 3 years. After that he went as Manager at Inchicore, the works of the Great Southern and Western Railway of Ireland, and in 1883 was appointed Locomotive Engineer of that company. In 1886 he was invited to become the Chief Mechanical Engineer of the Lancashire and Yorkshire Railway, and it was when holding that post that he laid out, equipped, and organized the shops at Horwich which were for very many years regarded as a model by all connected with such works. In 1899 he was appointed General Manager of the company, an unusual departure from custom but one that was fully justified by the success of his work in that position. One of many important works carried out under him as General Manager was the electrification of the Liverpool and Southport Railway, the first main line electrification in the United Kingdom. He was knighted in 1917. Sir John Aspinall resigned his position as General Manager and was appointed a Director of the company in 1919. In October of that year, however, he resigned his seat consequent upon his appointment as Consulting Mechanical Engineer to the Ministry of Transport. From 1924 until 1926 he acted as consultant to the newly-formed London and Midland Scottish Railway, retiring from active service in the latter year.

In 1898 he read a Paper [1] before The Institution on "The Friction of Locomotive Slide-Valves" for which he received a Telford Premium, and in 1901 he presented a Paper [2] on "Train Resistance" which attained a classic reputation and for which he was awarded the Watt Gold Medal. In December, 1936, he was awarded the Institution of Mechanical Engineers James Watt Medal for outstanding contributions to the advance of mechanical engineering. He delivered the James Forrest Lecture [3] in 1922.

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1 Minutes of Proceedings Inst. C.E., vol. cxxxiii (1897–1898, Part III), p. 13.

2 *Ibid.*, vol. cxlvii (1901–1902, Part I), p. 155.

3 "Some Post-War Problems of Transport." *Ibid.*, vol. ccxiv (1921–1922, Part II), p. 235.

Sir John Aspinall was elected an Associate Member of The Institution in 1881 and transferred to full Membership in 1887. He was elected a Member of Council in 1907, and Vice-President in 1916, and held the office of President in 1918.

He was also President of the Institution of Civil Engineers in Ireland in 1884–6, and President of the Institution of Mechanical Engineers in 1909. He was a Lieutenant-Colonel in the Engineer and Railway Staff Corps, a Knight of Grace of the Order of St. John of Jerusalem, a Knight of the Order of Leopold, and an Honorary Doctor of Engineering of Liverpool University.

He married in 1874 Gertrude, daughter of Mr. F. B. Schräder of Liverpool; she died in 1921. He leaves two daughters; his only son, Mr. J. B. Aspinall, who was City Remembrancer, died in June, 1932.

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